THE WEEK'S EVENTS

Wage Law Covers Some Contractors

Builders of interstate transportation facilities considered as being in interstate commerce

Although local construction contractors do not come within the jurisdiction of the federal Fair Labor Standards (wages and hours) Act of 1938, contractors who work on transportation facilities used in interstate commerce are probably subject to the act. Administrator Andrews of the Wage and Hour Division of the U. S. Department of Labor so held in Interpretive Bulletin No. 5 issued Dec. 2.

"Employees of contractors engaged in maintaining, repairing or reconstructing railroads, ships, highways, bridges, pipelines or other essential instrumentalities of interstate or foreign commerce," the administrator held, "would seem to be engaged in interstate commerce and subject to the act. And employees of contractors who are employed in repairing or altering buildings used to produce goods for commerce might be held by the courts to be engaged in a 'process or occupation necessary to the production' of such goods . . . and, therefore, within the coverage of the act."

"Employees of local construction contractors generally are not engaged in interstate commerce and do not produce any goods which are shipped or sold across state lines," it was held. "There may be particular employees of such contractors, however, who engage in the interstate transportation of materials or other forms of interstate commerce and are for that reason entitled to the benefits of the act."

The administrator is of the opinion that, in general, manufacturers or processors who produce goods for use wholly within the state but receive their raw materials from outside the state are not covered by the act, except as regards employees "purchasing the raw materials from other states or receiving them from other states."

In this as in previous executive bulletins, the administrator emphasized that the interpretation announced serves in general only to indicate the construction of the law which will guide him in the performance of his administrative duties until he is directed otherwise by the courts or until he changes his mind.

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Two Firms Share Work On Niagara Falls Bridge

Details of the division of engineering work on the proposed new Falls View bridge at Niagara Falls between the Edward P. Lupfer Corp. of Buffalo and Waddell & Hardesty of New York City (*ENR*, Dec. 1, adv. p. 25) are now available. The field work is to be carried out by the Lupfer organization, while the bridge designs are to be prepared in the Hardesty office.

Herlihy Bids Low On Chicago Subway

Bids opened Dec. 1 on first section of deep twin tubes in north State St.

The Herlihy Mid-Continent Const. Co., of Chicago, submitted a bid of $2,746,104 as the lowest of eight proposals opened on Dec. 1 for the first 3,600-ft. section of Chicago's new $40,000,000 subway. Bates & Rogers Const. Corp., Chicago, bid $151,000 higher; the other proposals ran up to a high of $4,536,792. This section, known as

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Wide World Photo

SIXTH AVENUE ELEVATED TO BE TORN DOWN

AFTER 60 years and a few months of service, train operation on the Sixth Avenue elevated line in New York City was halted at midnight Dec. 4 when the city took over the line from a bondholders committee of the Manhattan Railway Co. The line had been operated under lease by the Interborough Rapid Transit Co. The city acquired the line by condemnation for $12,500,000, of which $9,000,000 is a credit against back taxes due the city.

The next day the line was offered for sale as junk at public auction and a bid of $80,000 was offered by a West Coast firm, the Portland Mill Wrecking Co. If the bid is accepted, the company will have 90 days to remove the structure from the streets.

Built between 1876 and 1878, when operation started, the line passed through a number of hands and was acquired by the Manhattan Railway Co. In 1903, it was leased with the other elevated lines to the Interborough Rapid Transit Co. which has operated it up to the present.

The above view shows the elevated structure looking south along 6th Ave. from 54th St.

TVA Sells At Loss Morgan Says

Former chairman testifies that TVA could not meet expenses of private utility

Continuing its study of the power phases of the Tennessee Valley development, the joint Congressional committee this week heard further testimony by Arthur E. Morgan, deposed chairman of the TVA, and by Sherman M. Woodward, its chief water control planning engineer.

Topics discussed by Dr. Morgan included cost allocations among navigation, flood control, and power; wholesale TVA rates as compared with the cost of producing power; "yardstick" retail rates and their adequacy; and the power policy of TVA as at present conducted. His conclusion, briefly stated, is that present TVA rates, both wholesale and retail, are insufficient to return known costs and that TVA power is, therefore, being subsidized by the federal government.

David E. Lilienthal, he charged, had tried to force cost allocations on the basis of the "by-product" theory, by which only the expenses entirely due to power would be charged to it, omitting any share of commonly-used facilities. Morgan's refusal to accept this theory was responsible for the long delay in submitting the allocation report, which was finally submitted only after Congress had voted to investigate the authority last summer.

Power operations

Morgan presented detailed figures showing prospective annual losses on the basis of sales at the present TVA wholesale rates as compared with annual operating expenses (such as would be incurred under private operation) for the present three-dam, and the ultimate seven-dam, systems.

Essential figures from these estimated annual operating statements are:

Item	Three Dams
Operating revenues	$5,054,866
Operating expenses	1,371,000
Fixed charges:	
Taxes	252,743
Interest (3½%)	2,590,104
Depreciation	1,842,809
Development deficit	377,864
Total operating expenses and fixed charges	6,434,520
Annual deficit from operations	$1,379,654

These figures include the cost of interest during construction and organization charges—expenses, he said, that would necessarily be met by a private development. Brief comments on the bases of calculation follow:

Operating Revenues—Assuming the sale, at 60 per cent load factor and 10 per cent line loss, of output of the three dams at an average price of 3.23 mills, the 1938 (fiscal) average.

Operating Expense—Adds to the $1,096,000 reported by TVA as power operating expense in 1938 an additional $275,000, which is 40 per cent of the reported cost of water control operations. TVA does not charge this latter to power; Morgan believes power should carry part of the cost of reservoir operation, malaria control, and similar items and adopts the 40 per cent figure from the cost allocation report, which assigned 40 per cent of the cost of jointly-used facilities to power.

Taxes—Figured at 5 per cent of gross operating revenues, as directed in the TVA Act. Private utilities pay about 15 per cent; those selling only wholesale power about 25 per cent.

Depreciation—Straight-line method, at 2 per cent on dams, powerhouses and generating equipment; 4 per cent on transmission lines.

Development Deficit—This is an amortization over 25 years of the accumulated operating deficit during the load-building period, assumed as ending June 30, 1938.

Similar computations for the seven-dam ultimate system, scheduled for completion in 1942, indicate, Morgan says, annual expenses totalling $13,076,061; annual income of $9,919,209; and an annual deficit of $3,156,852.

Morgan also presented figures showing annual deficits even though interest during the construction period and development expenses were not taken into consideration, and another set on the basis of 3 per cent, rather than 3½ per cent, interest charges.

In closing this phase of his testimony the witness pointed out that over the past 50 years the trend of power generating costs has been steadily downward. There is no reason, he believes, to assume that this trend will suddenly stop while TVA pays off its obligations. Therefore, he believes, TVA will be forced to sell its power on a steadily falling market, while its annual costs will consist primarily of fixed costs that cannot be reduced and labor costs, that tend to increase.

The retail yardstick

Morgan then turned his attention to the retail "yardstick" rates charged by communities and associations distributing TVA power. These, he proved by direct quotation, have repeatedly been held out as demonstrations of what the true cost of electricity should be. Implied in the yardstick concept are two factors: that all costs of distribution be included in reported expenses, and that no element of subsidy enter into the picture.

These conditions, he charged, are not fulfilled in the TVA area. On the contrary, when the cost of many services rendered to these communities is taken into consideration, it is found that TVA is giving away its power and that the reported "profits" of the yardstick communities are really deficits.

Hidden service costs

The cost of many TVA services is not reflected in the accounts of the distributors, and much of it does not appear in the power accounts of TVA, he said—such items as rural load-building activities by land-grant colleges, grants and loans by PWA and loans by REA. However, reported load-building expenditures by TVA averaged over $3 annually per residential customer, whereas revenue per residential customer was only about $6 per year. In 1937 TVA promotion costs were $1.04 for each $1 of added revenue from residential sales; for 1938 the figure was $0.87. If other undisclosed costs were added, Morgan estimates that promotion costs would equal three-quarters of TVA's revenues from this business.

The witness also stated that, although TVA pays prevailing wages, the distributing communities do not do so; that wages paid average only 2/3 of the scale paid for comparable services by TVA itself.

It was also stated that the "yardstick" rates do not apply to many users of TVA current, since the distributing communities are permitted to make additional charges that raise the effective rate paid far above the yardstick level. There are two types of these: developmental surcharges, usually a 10 per cent addition to the monthly bill levied on all commercial and industrial customers; and an amortization charge, imposed on all classes of customers, usually 1¢ per kw.-hr. with a minimum of 25¢ and a maximum of $1 per month.

In closing, Dr. Morgan took occasion to restate his views on the TVA development as a whole. It has, he stated, wonderful possibilities. The plan of unified river development is a great advance over the old idea of piecemeal construction. Much of the criticism of TVA is not well founded: It is an experiment in a new form of government. Democracy is getting unwieldly and the TVA development may point the way to greater flexibility.

However, if TVA is to be saved, certain limitations must be placed on its administration. Government must be impartial; no matter whether a minority is popular or unpopular it should get a square deal. "That is what I have been fighting for."

Dravo Low Bidder On Delaware Tunnel

The Dravo Corp., Pittsburgh, submitted the low figure of $11,431,975 in bidding Contract 319 of the Delaware River Aqueduct tunnel, New York City water supply. The bids, opened Dec. 6 by the New York Board of Water Supply, covered 55,900 ft. of 13½-ft. tunnel between West Branch Reservoir and the Hudson River, to be driven from Shafts 7 and 8 already sunk under another contract. S. R. Rosoff was second low with a bid of $12,673,600. Six other bids ran up to $14,670,000, while the high figure of $19,019,075 was way out of line.

This is Dravo's first successful bid on the tunnel work, though the firm has just completed two contracts involving thirteen shafts on the aqueduct; it also built part of the Catskill Aqueduct about twenty years ago. When this section is awarded, all of the tunnel sections on the lower half of the project and east of the Hudson River will be under contract.

The board has awarded six contracts covering 36.3 miles out of the 85 miles of tunnel required for an aggregate amount of $76,027,249.

CONTRACTS AND CAPITAL

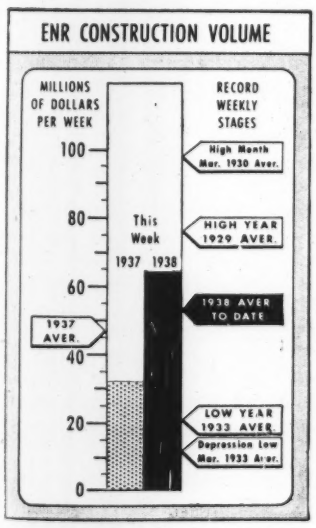
ENGINEERING construction awards for the week, $64,799,000, are 103 per cent higher than in the corresponding 1937 week but 5 per cent below last week.

Total construction volume for 1938 to date, $2,585,616,000, is 11.8 per cent above the volume for the 49-week period last year. This year's 49-week volume is 6 per cent higher than the total for the 52 weeks of 1937.

Private construction is 8 per cent above the 1937 week's total, but 30 per cent lower than a week ago. Public awards are 139 and 1 per cent higher, respectively, than last year and last week.

Public building awards, $25,300,000, are at their second highest weekly level of the year, and are largely responsible for the public construction gains. All classified construction groups except commercial building and large-scale housing gain over their respective 1937 weekly volumes. Waterworks, sewerage, industrial buildings, public buildings, earthwork and drainage, and highways report increases over the preceding week.

New capital for construction purposes for the week totals $76,264,000, an increase of 327 per cent over the volume for the 1937 week. This brings the new construction financing total for the year to date to $3,672,016,000, a 72 per cent gain over the volume for the 49-week period last year.



CONTRACTS

(Thousands of dollars)

	Week Ending Dec. 9 1937	Dec. 1 1938	Dec. 8 1938
Federal	$5,289	$10,133	$7,912
State & Municipal	17,853	44,457	47,451
Total public	$23,142	$54,590	$55,363
Total private	8,731	13,546	9,436
TOTALS	$31,873	$68,136	$64,799
Cumulative 1938 (49 weeks)			$2,585,616
1937 (49 weeks)			$2,312,143

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works, $25,000; industrial buildings, $40,000; other buildings, $150,000.

NEW PRODUCTIVE CAPITAL

	Cumulative 1937 49 Wk.	1938 49 Wk.
NON-FEDERAL	$1,387,585	$2,528,321
Corporate securities	501,780	372,209
State & Mun. bonds	477,740	683,500
PWA loans, grants	178,126	838,219
RFC loans	29,939	100,344
U.S.H.A. loans	.....	265,054
R.E.A. loans	....	67,495
Federal Aid-highways	200,000	201,500
FEDERAL	$746,223	$1,143,695
TOTAL CAPITAL	$2,133,808	$3,672,016

FHA MORTGAGES

	Week Ending Dec. 4 1937	Nov. 26 1938	Dec. 3 1938
Selected for appraisal	$8,359	$17,773	$20,000*
Cumulative 1938 (48 weeks)			$940,081*
1937 (48 weeks)			$565,484

* Subject to revision.

ENR INDEX NUMBERS

Index Base		1913	1926
Construction Cost	Dec.	234.89	112.87
Building Cost	Dec.	196.07	105.99
Volume	Nov.	199	87

Court Reinstates Ohio Engineer

Sparks, fired last March from bituminous engineer post, is ordered reinstated

Harry A. Sparks whose position as bituminous engineer with the Ohio department of highways was abolished last March, (*ENR*, March 10, 1938, p. 353) has been ordered reinstated in that position by the Franklin County Common Pleas Court, in an order handed down Nov. 29.

Sparks' position was abolished after he had testified before a committee of the Ohio Senate investigating graft in the highway department. Sparks had said that millions of dollars were being wasted annually because the highway department was paying excessive prices for hot mix roads.

A few days later Governor Davey told the state Assembly that Sparks was "inefficient, disloyal and contemptible" and that he intended to fire him.

When Sparks' position was abolished Highway Director Jaster said that this was done because of lack of work and as a "matter of more economical administration."

The court held that evidence indicated that Sparks' position was abolished because of his testimony rather than for reasons of economy.

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Ickes Issues Warning On High Prices

PWA allotments will be rescinded and the projects will be cancelled if contractors and material dealers ask unreasonable prices for construction projects between now and the first of the year, PWA Administrator Ickes warned last week. Ickes has received reports that in a number of instances bid prices on projects appear to have risen excessively, exceeding in many instances the money available for the project.

Because of the requirement that all projects be under construction by Jan. 1, it is impossible in most cases to readvertise for bids or to change plans and specifications to effect cheaper construction, Ickes pointed out.

Ickes said that reasonable increases in the price of materials and construction costs were taken into account when the cost estimates were made, but that "in a number of instances prices have jumped beyond all reason . . . I hope that industry will not, by its own tactics, defeat the very program which was set up for its benefit."

Road Service and Soil Stabilization Lead in Research Sessions

Annual meeting of Highway Research Board shows strong interest in earth roads and traffic problems

With a program expanded to occupy five days in place of the customary two, and with many duplicate or triplicate sessions, the Highway Research Board in its annual meeting at Washington last week recorded an attendance of about 500, largest in its history. Among the many subjects covered, earth road construction and traffic drew heaviest attendance; cross profile and roadsides also attracted active interest, while materials and design were less prominent than heretofore. A new departure in the Board's procedure was to give two of the five meeting days to soil problems. One group of sessions was devoted to soil mechanics problems, largely unrelated to roads.

Reporting on highway administration and finance, Thomas H. MacDonald, chief of the Bureau of Public Roads urged that road aid funds be allocated on a scientific rather than a political basis, and vehicle taxes be applied to a designated system of roads and streets.

Earth roads

Practice in grading soil mixtures for so-called stabilized soil roads, reported from a number of states, showed wide variation even between different soils in the same state. Grading specifications are used in most states, some based on a standard grading curve for material below No. 10 sieve and generally with exclusion of stone above 1 or 1½ in. size. However, J. L. Land said that in Alabama many successful materials fall outside the range of current specifications, and others referred to similar uncertainties in grading stablized or "earth mortar" mixtures.

For effective compaction the "optimum" moisture content was generally recommended, though determined by different methods. K. B. Woods reported that in Ohio, the required degree of compaction is controlled by the density of the material, from 90 per cent of laboratory compaction for soil weighing over 120 lb. per cu.ft., to rejection of soils weighing less than 100 lb.

Reports on the effect of admixtures in soil stabilization such as, salt, calcium chloride, tar, cement, asphalt, and emulsions, were uniformly favorable.

Nebraska has experienced much trouble from swelling subgrades, said G. F. Briggs, the pavement rising one to two inches at joints and edges; compaction after addition of sand aggregate reduced the trouble.

Practice in Denmark, as reported by Axel Riis, is to remove boggy material from the subgrade and replace it by an insulating layer of slag or other porous material.

Leo Casagrande, reporting on German practice, listed as important subgrade aids the practice of blasting marshy ground, placing a deep layer of cinders over fine-grained soil to prevent frost heave, and consolidating the ground around piles by electric current passed through aluminum strips, which in some instances triple the bearing capacity.

Control of frost heaving in Sweden, described by Gunnar Beskow, involves five methods used alternately or in combination: replacement of the heaving soil by better material, use of an insulating layer of peat moss or hay, use of a granular layer to cut off capillary water, deep drainage, and chemical treatment. New Hampshire has used chemical treatment successfully on fine-grained soil under flexible pavement, where drainage was ineffective, according to John Morton. In Michigan the use of a sand ballast layer below the pavement is now customary in heavy subsoil, O. L. Stokstad reported. Bailey Tremper stated that Washington has had success by using a sand cushion layer extending the full width of the embankment, draining blind springs and puddles in rock cuts, and keeping side ditches clear.

Discussion of chemical treatment to correct frost trouble indicated varying experience. Michigan has injected calcium chloride under concrete slabs without success, but found benefit in pouring the chemical into boils in gravel roads after opening them up with a posthole digger. Several other states reported that stabilized gravel roads chemically treated withstood frost damage better than untreated roads of similar type.

Careful attention to road cross-section, lane layout, shoulders and roadside treatment was evident. W. H. Simonson, reviewing current practice in layout of multiple-lane roads, found that less than 20 per cent of roads with four or more lanes have divided roadways. George E. Hamlin described the comprehensive work done by Connecticut in building hard shoulders on its roads, to widen the travel width.

Massachusetts, according to George H. Delano, now plans the development of the right-of-way more or less completely, all designs being reviewed to provide for utilities, picnic areas, turnouts and sidewalks. Construction of several hundred miles of sidewalk in the past three years has cut pedestrian accidents in half. Murray B. Van Wagoner reported that roadside development in Michigan increases safety by giving fatigued drivers opportunity for rest and by outlining the road at time of low visibility.

Road service and control

Traffic problems gave rise to an extended symposium. The better safety record of 1938 as compared with 1937 was variously attributed to better cars, better drivers, and chance. Road signs are becoming so numerous as to be unsafe, and some states are eliminating the less important signs. Turnouts for trucks were suggested as proper on long grades. Study of intersection hazards in relation to angle of entrance, widening and signaling was reported; one state has found that four times as many accidents occur at a cross-road as at a T, V, or fork intersection (relative volumes of traffic not reported).

Preliminary results of highway capacity studies were presented by O. K. Normann, of the Bureau of Public Roads. They indicate that the maximum number of vehicles passes at speeds between 30 and 35 m.p.h., and that the time interval between vehicles under capacity conditions is 1½ sec.

Measurement of truck performance on grades was described by C. C. Saal, from the National Highway Planning Survey work. Methods are still in course of development. Delays caused by trucks on narrow hills can be remedied, he stated, by reducing the grade, adding a supplementary lane, increasing motor power, or adding distance.

Materials and construction

Numerous new facts on steel, brick, concrete and bitumen were reported. Prof. H. J. Gilkey gave results of bond tests confirming earlier findings that first slip occurs at about the same stress for all types of rod reinforcement. Prof. T. V. Mylrea showed from results of comparative impact tests on concrete beams with structural and high-tensile reinforcement that the latter type is fully as resistant to shock fracture as the former. Incidentally, rupture impact was found to be 25 to 30 times as great as yield-point impact, demonstrating great reserve of strength in under-reinforced beams.

Dowel action at slab joints, studied

DAM NAMED FOR ENGINEER

CALEB MILLS SAVILLE, manager and chief engineer of the Hartford, Conn., water bureau was honored by the Metropolitan Water Supply District when it gave his name recently to the dam it is building on the East Branch of the Farmington River.

Largest dam in Connecticut, Saville dam is scheduled for completion the first of the year. It is a 137-ft. high earthfill embankment with concrete corewalls and a 200-ft. spillway. The dam will create a 2,320 acre reservoir with a water yield of 54 m.g.d. Work on the dam started in 1933 and when completed will cost about $2,850,000 exclusive of land, reservoirs and highways.

by B. Friberg, is proportional to yield-point of steel and strength of concrete. The test indicated 1 in. dowels to be adequate for usual loads and pavements and ¾ in. dowels for light loading.

Prof. F. C. Lang offered a beam durability test of deleterious matters in aggregate as preferable to direct freezing-and-thawing tests of aggregate.

Brick and asphalt

Progessive improvement in asphaltic fillers and cushions courses for brick pavement was reported by Prof. J. S. Crandell as the result of service trials in Ohio. A straight asphalt-base material has given best service to date, showing no exudation. Shale roadbed construction in Pennsylvania has been successful, according to G. A. Rahn, because of careful selection of the shale and use of a full-width roadbed with feather edge at the ditch line, laid on a well-crowned subgrade and surfaced with 2-in. bituminous mat. A 6-in. rattler of Los Angeles type was designed to test shale for acceptability.

Prof. M. G. Spangler reported tests of wheel-load distribution through a bituminous pavement slab, in which the pressure intensities were measured by steel tapes sliding between inclosing steel strips. The distribution was found directly proportional to thickness of slab (at 70 lb. tire pressure the max. pressures under the wheel were 65 lb. per sq. in. for 3-in. slabs and 13 lb. for 15-in. slabs).

A joint committee on construction equipment of the board and the American Road Builders Association, formed two years ago, reported on current practice in mixing and laying stabilized earth road material. It also studied problems of mixing, vibration, and placing of pavement concrete. The committee discussion indicated that efficient use of equipment is sometimes rendered difficult by extreme specifications, such as the requirement of slumps below 1 in. for concrete to be placed with vibration. Under these conditions mixer and bucket discharge and proper distribution of the concrete across the slab width have proved to be difficult, and segregation is increased. The addition of transverse distributors to mixers was suggested as a means of improving the slab concrete under such specifications.

A series of sessions on soil mechanics included some pertinent facts on fill construction and its relation to retaining walls and abutments. T. E. Stanton reported that compaction of road fills has been regular practice in California for several years and that fills up to 110 ft. high, properly compacted during construction, show little settlement; field compaction of 90 per cent of laboratory compaction under a pressure of 2,000 lb. per sq. in. is specified. Slips and settlement arise chiefly from foundation failure.

Other discussions in soil mechanics dealt with embankments, earth pressure, foundation settlement, pile buckling and the measurement of consolidation and shear strength. Much divergence of opinion on methods and value of shear testing was apparent.

Intrastate Utility Subject to NRLB

Supreme Court upholds jurisdiction over Edison Electric but upholds AFL contract

Even though the Consolidated Edison Co. of New York sells power only within the state of New York, the company is nevertheless subject to the provisions of the National Labor Relations Act because it sells its power to railways, telegraph and radio corporations and other interstate facilities. The Supreme Court so held in upholding the jurisdiction of the National Labor Relations Board over employee relationships of the company. The board had acted on a complaint brought by a CIO union.

At the same time, the court held that NRLB had acted improperly in ordering the company to abrogate a contract it holds with the International Brotherhood of Electrical Workers, an AFL affiliate.

Interstate use of power

Pointing out that power sold by the utility was used in interstate commerce, the court said, "It cannot be doubted that these activities, while conducted within the state, are matters of federal concern. In their totality they rise to such a degree of importance that the fact that they involve but a small part of the entire service rendered by the utilities in their extensive business is immaterial in the consideration of the existence of the federal protective power."

The court upheld a number of orders issued by NLRB directing the company to cease certain unfair labor practices, including an order to stop recognizing the IBEW as exclusive bargaining agent for all employees; the court held that the AFL union was bargaining agency only for its own members in the company's employ.

Procedure incorrect

NLRB had contended that the AFL union had been granted a contract only "as a device to consummate and perpetuate the company's illegal conduct." The court, however, said that the board had not demonstrated this conclusion and that "if the board intended to make that charge, it should have amended its complaint accordingly, given notice to the brotherhood, and introduced proofs to sustain the charge.

"Instead, it is left as a matter of mere conjecture to what extent membership in the brotherhood was induced by any illegal conduct on the part of the employer."

Philip Hart Dies in Plane Crash

Allan W. Cuddeback, New York waterworks man, dies at the age of 70

Philip Hart, president of the Pacific Bridge Co., San Francisco, was drowned Nov. 29 with other passengers on a Portland-San Francisco airplane that made a forced landing on the ocean near San Francisco. Born 51 years ago in Portland, Ore., he was graduated from Massachusetts Institute of Technology in 1910 and then engaged in engineering and contracting on the West Coast.

His firm completed the difficult construction of the south pier of the Golden Gate bridge and built, or had a part in building, piers for other notable bridges including the Burnside and Ross island bridges at Portland, the Columbia River bridge at Longview and the west crossing spans of the San Francisco Bay bridge. At the time of his death he was returning from supervising the start of work on the Narrows Bridge at Tacoma, Wash., for which his firm had just received the contract.

ALLAN W. CUDDEBACK, who had been president of the New York Water Service Corp. since 1931, and who was president of the American Water Works Association in 1926, died in New York last week at the age of 70. Following his graduation from Stanford University in 1894, he worked with the Coast and Geodetic Survey and other surveys, then went east to do water supply work in the New York metropolitan area, chiefly in New Jersey. From 1926 to 1933 he was vice president and general manager of the Federal Water Service Corp.

JOSEPH ROSENTHAL, 72, Brooklyn contractor who built several links of the New York Independent subway system, died last week in Brooklyn.

R. L. LATHAM, 61, cheif engineer of the Toronto, Hamilton & Buffalo Railway, died recently in Hamilton, Ont. He became assistant engineer.

HENRY LOHSE, SR., 66, Seattle contractor, died in that city Nov. 28. With three partners, he formed the Sound Construction & Engineering Co. at the turn of the century.

MAJ. JOHN A. WILSON, engineer and adjutant general of Puerto Rico for 20 years, died Dec. 2 at the age of 76. As one of the original partners in the engineering firm of J. G. White & Co., New York, he participated in the construction of some of the country's early utilities before going to Puerto Rico in 1898 to electrify the old Spanish steam tramway which served San Juan.

OTHMER WALLACE ROSS, member of the engineering staff of the Welland Ship Canal, died recently in St. Catharines, Ont.

WILLIAM P. MOLIS, 83, veteran superintendent of the Muscatine, Iowa, water department, died there Nov. 27.

MILTON C. DAVIS, 78, Baltimore general contractor for 45 years, died there Nov. 30.

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Herlihy Offers Low Bid On Chicago's Subway

(*Continued from page 715*)

Contract S-5, will be twin horseshoe tunnels with monolithic concrete lining and common center wall (*ENR* Nov. 17, 1938, p. 602) extending from Illinois St. to Oak St. in north State St. The inside clearances of each tube will be 15 ft. 9 in. wide and 19 ft. high above base of rail. Two 500-ft. stations are included at Grand Ave. and Chicago Ave.; station platforms will be built in smaller tunnels outside of the railroad tunnels.

This part of the subway lies in typical Chicago blue clay; base of rail is 40 ft. below street level. Construction will be by the liner plate, steel rib and needle beam method with the concrete lining kept close to the face. Driving procedure, under a minimum of 3 lb. compressed air, will be similar to that used on large intercepting sewer tunnels recently built by the Sanitary District of Chicago.

Inasmuch as the Chicago subway contracts must be completed in eighteen months a high-speed construction program has been developed by the newly formed Subway and Traffic Commission. Short contract sections will permit work to proceed in many places at the same time; bid openings are scheduled three weeks apart with work on the last section to be started by July, 1939. Thereafter bids on station contracts are to be called for, with the last scheduled in August.

The river crossing will be carried out by the same method as was used in constructing the La Salle St. street car tunnel more than 25 years ago, floating into place and sinking a double steel tube into an excavated trench and connecting up the ends with cofferdams on each side of the river.

New York cut-and-cover methods will be utilized in building the mezzanine, station stairways, entrances, and connections; openings will be 18 to 20 ft. deep and 60x80 ft. in area.

Seattle Pontoon Bids Are Rejected

Bids on four units of Lake Washington project exceed estimates and are thrown out

The Washington State Toll Bridge Authority last week rejected as too high bids on four of the eleven units involved in construction of the $8,000,000 Lake Washington pontoon bridge project at Seattle. The authority will open new bids Dec. 20 for the pontoon bridge itself, the west and the east approach, and for a bridge over the channel separating Mercer Island on the east from the mainland.

Specifications for the pontoon section have been revised somewhat to provide for the use of steel instead of concrete on the transition sections, structural steel ties instead of concrete on the center movable span, and for a change in the under-water depth to permit the construction of the pontoon section at places other than on Lake Washington.

The only bid received for the Pontoon structure was made by the General Construction Co. of Seattle and the Columbia Construction Co. of Bonneville, bidding jointly at $3,795,000. The estimate on this unit was $2,500,000. No individual bids were received on the other three units, but the Chicago firm of Bates & Rogers offered to build the east channel bridge and the two approaches for $1,500,000. The total estimate on these units was $1,300,000.

Bids on the other units of the project, which are either within or near the estimate, have been held until new offers are received on the major units.

Approach work legal

Earlier in the week, a possible legal obstacle to construction of the project was set aside when the state supreme court ruled that the 1937 state legislature intended to give the toll bridge authority broad powers to build tunnels and highway approaches to the bridges it builds. The state auditor had refused to honor warrants for payment for preliminary work on the tunnel and approaches, claiming that the work "is too remote from the original structure to be properly construed as part of the approach to the proposed bridge.

The court declared that it "will not concern itself with the exercise of discretion delegated to an administrative agency unless it appears the agency is acting in bad faith. . . . It must be borne in mind that the bridge will be a toll bridge, and in order to attract and divert highway commerce over the bridge, it is essential that the channels of travel be made readily accessible to the bridge users."

COMMENT AND DISCUSSION

Readers' opinions on matters that concern the engineer

Engineering Students

Sir: My interest has been aroused by a letter appearing in your issue of Oct. 20, page 483, under the heading "Engineering Students". As a graduate civil engineer with several years of engineering construction experience, and as one at present engaged as the director of the placement bureau of a large eastern university, I have some understanding of, and much sympathy with, the problem which has faced our civil engineers since 1930—that of securing professional employment in line with their abilities and training, and of securing proper recompense for professional services rendered.

Unfortunately, the old law of supply and demand is still operating. Until there is a genuine "building boom" that Utopia of the construction engineer which seems always to be just around the corner, there will continue to be capable men receiving inadequate salaries, other capable men in inferior positions, and some with empty pockets, no jobs, and most disheartening problems. The answer for many must be a courageous and imaginative effort to redirect their energies to some other field of work. One can have nothing but admiration for Mr. Wherry and all those others who have kept their chins up, doing anything available from ditch-digging and door-to-door canvassing to engineering and back again to keep body, soul, and home together.

Mr. Wherry's statement that graduates of colleges having placement bureaus must continue to pay dues to alumni associations or be deprived of the use of the placement bureaus, demands some correction for the benefit of those many engineers who graduated before the days of such services by their Alma Maters and on behalf of those colleges and universities which do not conduct their placement services in the above manner.

There are many institutions, including that which I represent, which make no direct or indirect charges for the placement service which they provide. Such institutions feel that the expense entailed is justified and that the furnishing of employment advice and information to alumni is a proper function of the college or university. Other institutions, unable to bear the whole expense of such a service without alumni support, charge fees for registration or placement, or require membership in their alumni organizations. Many of this latter group would gladly remove all charges if it were financially possible.

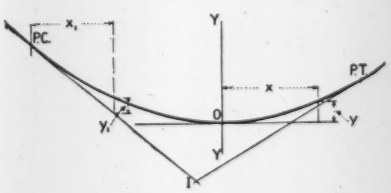
I question Mr. Wherry's suggestion that he spent all his capital just to make a certain school his Alma Mater. I believe he spent his money to get an engineering education, and I would venture to suggest that he got his money's worth before placement entered the picture.

The great increase during the last six years in the number of colleges and universities providing some form of placement service is evidence of their desire to assist the young alumnus, even though such assistance cannot always be offered without some charge.

HERBERT H. WILLIAMS
Cornell University
Ithaca, N. Y., Oct. 24, 1938

Parabolic Highway Curves

Sir: In *Engineering News-Record* of Oct. 27, Donald Bregman comments on the subject of "Fitting a Parabola to Grade Tangents," and seems to have based his analysis on assumptions that are not altogether justified. His analysis seems to apply to a parabola so placed that its vertex, or point of maximum curvature, comes at the P.C., thus requiring that the axis of the parabola be tilted in order to produce a condition of tangency at this point. This tilting of the axis of the parabola causes the mathematics to be somewhat more involved than would otherwise be necessary.



Regardless of the grades of the approaching tangents it would seem to be preferable to connect them with parabola having a vertical axis and therefore having its vertex at the low (or high) point. While the equation of this parabola, $y = k\,x^2$, applies primarily to the vertical and horizontal distances from the vertex, it can readily be interpreted to apply as well from any other point on the curve. Thus, if we know a point on the parabola such as the P.C. and the slope of the tangent at this point, then at any distance x_1 measured horizontally from this point the vertical offset between the curve and the tangent is expressed by the equation $y_1 = k\,x_1^2$, which gives a curve of the same shape as if we were working from the vertex and using the same value of k. Such a curve therefore can be computed from any tangent, or part of it can be computed from one tangent and part from another.

THOMAS C. COLEMAN
West Somerville, Mass., Nov. 12, 1938

Chemical Consolidation

Sir: We have read with interest the authoritative articles by Mr. Gutmann on "Algerian Rockfill Dam Substructures", particularly the one which appeared in your issue of May 26. We were however surprised to read on page 750 that the process of chemical injection used was "generally known as the K.L.M. process."

As inventor of this process, patented in the principal countries, and engineer-consultant of the Societe Sondages - Etanchements - Consolidations in Paris which executed the work, I must confess that I have never heard this process designated by the letters K.L.M. and I don't know what they mean.

All published accounts of this work refer to it as "the RODIO or GEL RODIO process of chemical injection."

We should be grateful if you would to good enough to call this point to the attention of your readers by publishing a note of correction in a subsequent issue.

GIOVANNI RODIO
Milan, Italy, Oct. 25, 1938

ENGINEERING NEWS-RECORD

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Editorial Offices at 330 West 42nd Street, New York

DECEMBER 8, 1938

ANY DOUBT about the wide range of present-day road problems or the intensity of interest and effort that is being applied to their solution is dispelled by such a meeting of road engineers and students as that of the Highway Research board last week. Crowded meeting rooms and close attention to the reports of experience and opinion characterized the situation even better than the program makeup. Measured by this standard of interest, operation and service experience took the lead over design and construction procedure, for travel speeds, intersections and traffic circles furnished even more active discussion than roadbed structure and surface treatment. Judging by such evidence of present trends it seems reasonable to believe that by the time the current highway planning surveys are completed technical study will be concentrated emphatically on road service as a necessary auxiliary to both general plan and detail design. Since structural problems will not decrease in importance meanwhile, the dimensions of road study are certain to see a marked increase.

Rise of the Earth Road

EQUALLY EVIDENT in the Research Board meeting was the prominence of earth-road problems. Soil stabilization appears to be a word to conjure with, for the time being, and the many unanswered questions that lie in this type of construction give assurance that future technical meetings will also find much of their time occupied by earth-road discussions. Some developments suggest, however, that the original term "low-cost roads" no longer can be applied generally to modern earth roads, for some of the construction procedures used in building stabilized soil roads are fully as complex and costly as those required to lay a concrete slab pavement. This development compels thought because it departs so far from the important low-cost objective which brought about the current interest in earth roads. Ingenuity and study should be invoked to show a way back to the low-cost concept, or else we will be in the absurd position of combining high construction cost with high maintenance cost. The situation forecasts extensive further growth of investigation and practice in the application of local materials, from the shale rock roadbeds of Pennsylvania to the soil concretes of the North and topsoil surfaces of the South.

Machines to the Fore

FINALLY the week's developments in road study show that the machine is at last to have its day. Equipment has been placed under study with respect to efficient use, adaptability to changing requirements and coordination of machine performance with design requirements. It is an innovation of substantial promise. The keen practical thinking that has been given to road problems by the builders of machines should now have opportunity to exert its deserved influence on road engineering procedure, and in turn to be influenced by it in the direction of more uniform execution of design essentials. The gain that may be expected from better coordination of equipment and design is difficult to forecast, but in ultimate effect it undoubtedly will mean much for the efficiency and economy of roadbuilding. The machine can always do more, do it better and do it cheaper than shovel or pickax if designed with understanding and skill.

Sixty Years' Life

WRECKERS bought the Sixth Avenue Elevated structure at auction in New York last Monday, and so a sixty-year career of essential public service of an urban transportation artery comes to an end. New and revolutionary in its day, the elevated railroad made metropolitan growth possible, and expanded many-fold the limits set by omnibus, horsecar and electric railroad. Yet we now see it obsoleted by progress, its place taken by more modern and more costly devices, leaving its defects as its sole remaining distinction. If even so necessary a service agency can survive only sixty years, must we not expect that many other modern works will have a strictly limited life? It may be prudent to keep the possibility in mind and to be cautious about premising financial or other undertakings on the assumption of indefinite existence.

Chicago Drives Ahead

A PHENOMENAL engineering performance is revealed by the opening of bids for the first Chicago subway section on Dec. 1—the creation in less than three months of a complete plan for a subway system, including both general scheme and working details of construction. When the PWA Advisory Board reported on the project, only a few months ago, it recommended changes that threw the original project overboard almost in its entirety. The shallow four-track line was replaced by dual two-track lines of deeper tunneled construction and in large part on changed route. This meant complete

redesign, a task that made abnormally severe demands on the local design organization. So rapidly has the work been carried out that it was possible to ask for bids on the first section by Dec. 1, and there is definite prospect that work can be started before the end of the year. The Chicago engineers deserve credit for a rare achievement. Their success brings into somewhat nearer prospect the seemingly impossible performance of building the whole of the subway system in 18 months' time.

Scope of Wage-Hour Act

EARLY AND UNOFFICIAL statement of Administrator Andrews as to the effect of the Fair Labor Standards Act on construction indicated that most construction work is outside the scope of the act. The wage-hour act applies to employees "engaged in (interstate) commerce or in the production of goods for (interstate) commerce", and Mr. Andrews took the position that dams, bridges, or buildings obviously are not "goods for interstate commerce". This view still is held by Mr. Andrews and the staff of the Wage and Hour Division of the Department of Labor, but it is so rapidly being limited by qualifications as to make it necessary for both contractors and engineers to follow closely the current developments with respect to the act.

The first definite limitation is found in section 12 of Interpretative Bulletin No. 5, just issued by the Wage and Hour Division. This section states that, though contractors on local construction operations are not subject to the act, yet their employees who engage in moving material to the job are under the act if the movement is interstate in character.

A more important broadening of the scope of the act as it affects construction is found in Section 13 of Bulletin No. 5, which states: "Employees of contractors engaged in maintaining, repairing, or reconstructing railroads, ships, highways, bridges, pipe lines, or other essential instrumentalities of interstate or foreign commerce would seem to be engaged in interstate commerce and subject to the act." Mr. Andrews is not willing to go farther at the moment, but states that courts may hold that the act applies to employees of contractors who are employed in repairing or altering buildings used to produce goods for commerce.

This interpretation of the act also would seem to put all contractors on street paving work within the act, and presumably all kinds of river and harbor work. Consequently, a large part of all construction is under the act, according to the Andrews view.

Many engineers have asked whether consulting engineers come under the act. As yet Administrator Andrews has made no ruling on that point. However, Bulletin 5 appears to indicate that the employees of all engineers who are doing work outside their own states come under the act. The bulletin says that "it seems clear that the term 'goods' includes publications, pamphlets or any other written material," also that the act applies to "research and compilation services." Read in connection with the ruling as to what construction work is under the act, these interpretations make it clear that the employees of a large number of consulting firms also are under the act.

In issuing all his interpretations, Administrator Andrews is careful to point out that these are not binding, that the only way in which interpretations which go beyond the clear wording of the act can be made binding is to have a court make them. The courts, however, are continually widening their interpretation of interstate status, as shown anew by the Supreme Court's decision of Monday in the Consolidated Edison case. Under these circumstances contractors need to recognize that to go contrary to Mr. Andrews' interpretations may involve them in costly and probably fruitless litigation.

Information, Please

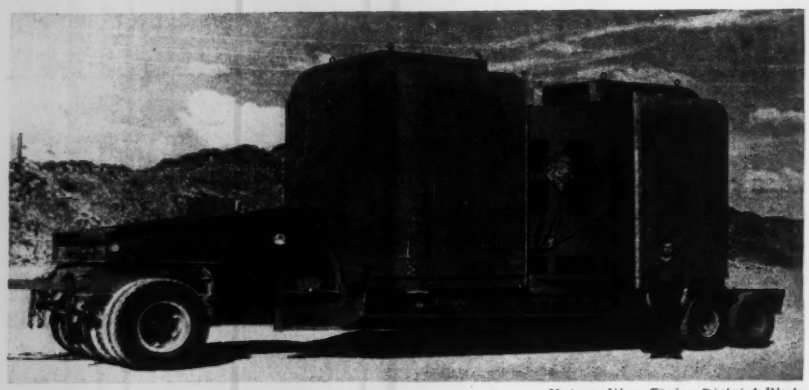
ADDRESSING itself to municipal budget matters, the Baltimore *Sun* asks, among other things, "How much of the vast Department of Public Works is absolutely necessary? . . . A little specific information on specific municipal undertakings would seem to be the need of the moment." Therein lies a challenge not only to Baltimore but to municipalities—a strong program of public relations is called for.

The citizens, for whom the *Sun* spoke, have been permitted to remain in ignorance of important public activities. And quite naturally when the time comes to allot money they question the "necessity" for public-works facilities and services. These essentials of community life are taken so much for granted that the public easily becomes apathetic to their existence—that is, until something goes wrong or the budget is under scrutiny. Then the administrator is called up to defend his department and the work it does. But it is then too late to "sell" public works. The only effective "selling" job is done by constantly keeping the public interested and informed—educating them, so to say, regarding the services they receive.

Baltimore, whose public works department is capably staffed and administered with technical competence, is not unique in the relation of its citizens to public improvements. There are few cities in which it can be said that public interest and support have been adequately cultivated. To continue to neglect this vital prerequisite to successful administration is to perpetuate misunderstanding, ignorance and prejudice.

PICTURES IN THE NEWS

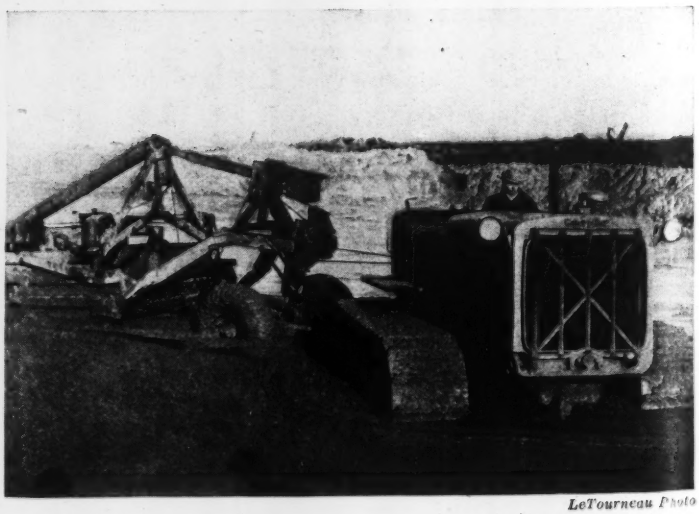
SEWAGE PLANT? No, it's a monitor roof on Industrial Rayon Corporation's new plant at Painesville, Ohio. Over 371,000 glass blocks were used in construction of this windowless plant. A general view appeared in ENR Nov. 24, 1938, p. 630.



Metropolitan Water District Photo

HEAVY HAULING. A capacity load for the trailer, this mass of metal (and the insulation, etc., that it conceals) is part of an electric motor for the Eagle Mountain pumping plant on the Colorado River Aqueduct of the Metropolitan Water District of Southern California.

PEA SOUP. England is the battleground where these and other pieces of equipment are striving to clear out a soggy earth bowl in construction of Staines Reservoir near London. Mud-removal with American equipment such as these shown compared favorably with removal by pumping.



LeTourneau Photo

FIG. I. NEW LABORATORY AT ST. ANTHONY FALLS ON THE MISSISSIPPI RIVER

New Hydraulic Laboratory at St. Anthony Falls

University of Minnesota uses abandoned power site for an unusual laboratory for hydraulic research and testing of hydraulic equipment

A NEW hydraulic laboratory for the University of Minnesota was formally dedicated on Nov. 17. It is located on Hennepin Island at St. Anthony Falls in the Mississippi River utilizing early water rights and the site of an abandoned Minneapolis water pumping station.

The laboratory facilities are grouped essentially into five units; the main experimental laboratory, the hydraulic machinery laboratory, the turbine testing laboratory, the large-scale volumetric measuring tanks and the administration and lecture rooms.

The main experimental laboratory is 300 ft. long and 45 ft. wide. It is two stories high and contains three large channels extending the entire length of the structure. One channel is an overhead flume 8 ft. wide and 9 ft. deep, connected directly to the headwater above the falls, and is provided with numerous offtakes to supply water for the various experimental projects. The others are low-level channels below the level of the main floor. Of these one is a waste-way and the other an experimental flume arranged for a wide variety of experiments. The latter is 9 ft. wide and 6 ft. deep and is supplied directly from the upper pool of the river through a pressure tunnel. Enough head is available to put water through the flume at the rate of 35 ft. per second for shallow depths. A towing car will make it possible to pull current meters, model ships and the like through the flume with the water either at rest or in motion.

Turbine testing facilities

The hydraulic machinery laboratory also has a clear height of two stories and is 34 ft. wide and 25 ft. long. It is equipped with an overhead crane. At one end of this laboratory there is a penstock shaft, 20 ft. square and 30 ft. deep, below the machinery testing floor; the shaft provides a means of bringing the water from the overhead channel to the turbine testing laboratory, the floor of which is 46 ft. below the headwater pool.

The turbine testing laboratory adjoins the hydraulic machinery laboratory but at a lower level. It is of irregular shape in plan, two sides being formed by the limestone ledge of St. Anthony Falls. A tailrace channel traverses the length of the laboratory beginning in the penstock shaft and extending to the tailwater pool below St. Anthony Falls.

The volumetric measuring basins are constructed with their bottom just above the tailwater pool. They are so located that the flow from all laboratories except the turbine testing laboratory can be measured. A central control house is arranged to operate large cylindrical valves in a diverter system for the tanks, also in the tanks themselves. Recording and indicating gages are located

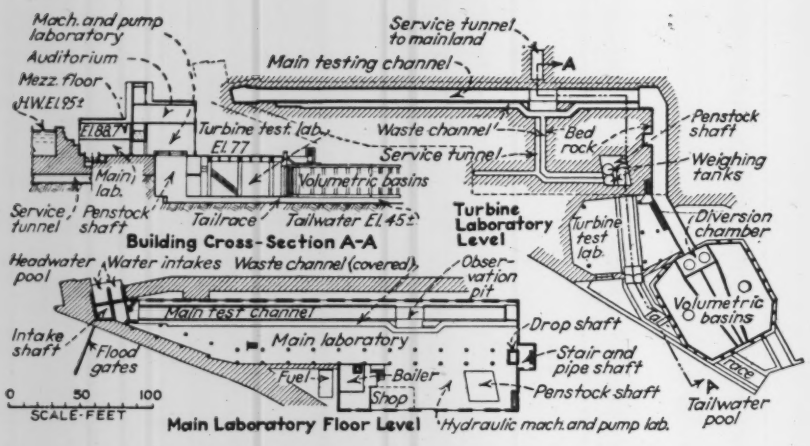


Fig. 2. Major facilities of the laboratory are at three levels, all below the upper pond level and arranged to take advantage of the site. Two of the principal levels are shown here.

in the control house. The valves are laid out to operate pneumatically. The measuring system is designed to handle a continuous flow up to 300 sec.-ft. It is intended to use this discharge measuring arrangement for check measurements on large-scale experiments and for calibrating water-measuring devices to be used in connection with the turbine testing laboratory.

Unique lecture facilities

Administration and lecture rooms are provided in a superstructure above the hydraulic machinery laboratory. A unique feature here is a demonstrational lecture room so arranged that large quantities of water can be readily handled in various types of experiments. Below the lecture platform is the main overhead supply flume for the laboratories while above the lecture platform is a head-control room containing a constant level reservoir. At one side of the lecture platform is a stairwell and pipe shaft providing access to the laboratory below and control room above; at the other side an apparatus room is arranged for housing various pieces of equipment.

In developing the site some 30,000 cu.yd. of bed rock and large boulders were excavated in order that the operating floors would be materially below the level of the river above the falls, so that part of the river might be diverted directly through the laboratory for use in experiments without the necessity of pumping. Close regulation of both the headwater and tailwater levels of the Mississippi River at the falls provides a head of 48 to 50 ft. at the laboratory at all times. Water rights of very early priority were acquired from the city permitting a flow of 40 sec.-ft. through the laboratory at all times, and larger quantities with higher discharges of the river. (The water rights attached to the site were utilized in early times for a saw mill; they were later acquired by the city for the operation of water turbines directly connected to a pump delivering a municipal water supply.)

Lorenz G. Straub of the College of Engineering and Architecture, University of Minnesota, is consulting engineer for the project.

Though the laboratory was not completed until fall, construction was started in June on a large river analysis project to be carried on by the engineers of the U. S. Army in cooperation with the university staff. It involves a working model of the projected navigation development over St. Anthony Falls on the Mississippi River in Minneapolis. The scale is 1:50 and the completed model is 36 ft. wide and 160 ft. long. Flow through the model will be directly from the Mississippi River diverted from the upper pool and discharged into the lower pool.

Several other projects, already under way in the new plant, include a study sponsored by the American Society of Civil Engineers concerning sedimentation at the confluence of river channels. Other studies similarly sponsored deal with seepage through sand dams, also a study of viscous flow in open channels.

Wisconsin Regulates Well Drilling

THE Wisconsin well-drilling sanitary law passed in 1936 is now in full operation, after two years of preliminary work in organizing its administration and developing support of the public and the drillers in its enforcement. It is administered by the State Board of Health through its Bureau of Sanitary Engineering. As explained in an address by Louis T. Watry, superintendent of well drilling for the Board of Health, every man who engages in well construction (except the driving of well points) pays a registration fee of $10. The law requires that all wells must meet specified minimum standards of construction which have been drafted by the state authorities. For each well the driller must file a report, including a log of the drilling and a plan of the location, and must also submit a sample of the water.

Enforcement has been very lenient, but a start in more strict enforcement has been made in the regulation of wells drilled for schools and for federal housing projects. Last year, there were 16 arrests; in 14 cases the men paid fines and registered, one served a term in jail and one quit the business. The law provides for fine and imprisonment and also for suspension or revocation of the permit to drill. One difficulty has been the slowness of the drillers to cooperate with the authorities charged with administration of the law. In Illinois there has been some agitation for a similar law, but no definite steps have been taken.

• • •

Sewer Rentals for Revenue

Several hundred cities in the United States make special rental charges for use of their sanitary sewers. Among the varied types of rate base used and for rental charges are: number of plumbing fixtures, metered water consumption, number of sewer connections, type of property, and number of persons.

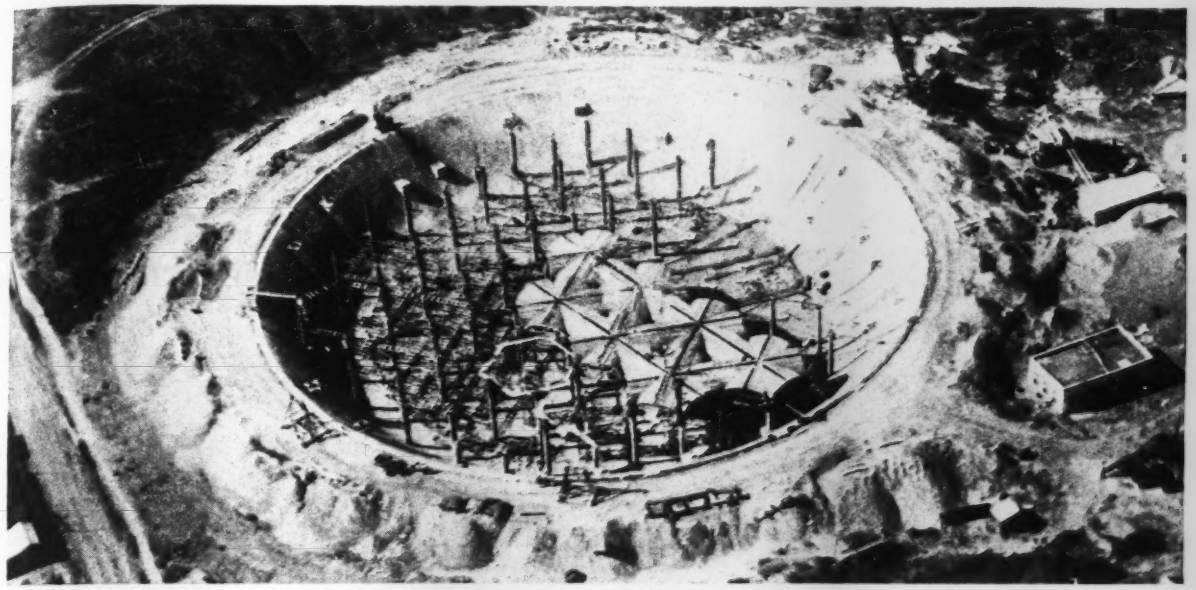


FIG. 1. BOWL-SHAPED RESERVOIR COVERED WITH THREE-WAY GROINED ARCH ROOF

Corpus Christi Builds a Covered Reservoir

CHARLES TERRELL BARTLETT
Consulting Engineer, San Antonio, Texas

Unusual groined arch roof and difficulties with leakage through the lining of the water reservoir at Corpus Christi give the project special interest

AN UNUSUAL groined arch roof and trouble with excessive leakage through the lining of the recently built water storage reservoir at Corpus Christi give that structure unusual interest for both designers and operators of water plants.

The reservoir is of 10 m.g. capacity, a bowl shaped, concrete lined basin in which sand and clay taken from the central excavation were used to build the encircling embankment. The roof is of reinforced concrete, of groined arches formed by the intersecting barrels of three series of arches set at 60 deg. angles.

The reservoir is situated within one half mile of the business center of the city and provides a much needed reserve and equalizing supply to augment the 750,000 gal. stored in elevated tanks within the city. The city's supply comes in a 15-mile line from a filtration plant at Calallen on the Nueces River.

The central portion of the reservoir bottom is 91.3 ft. in radius sloping down 3½ ft. to the tank center. A 30-ft. radius curve in vertical radial planes connects the conical bottom with the slope walls, which are on a 45-deg. incline. The water depth at the center is 34 ft. and the inside top diameter of the lining 266 ft. The floor lining at the center is 19.5 ft. below original ground level and the remaining 15.5 ft. of the water depth on the average is created by the embankment.

The radius of the rim accords with the hexagonal column spacing bringing the outer columns of the fourth ring of hexagons surrounding the central hexagon 10½ in. outside the top of the slope wall with these outer column faces flush therewith. The columns are hexagonal with 12½-in. faces, reinforced in the case of the taller columns with six 1-in. round bars, and with ⅜-in. spiral stays on 6-in. pitch. The columns over the bottom area and the lower half of the slope wall are carried on footings independent of the lining. These footings are 8 ft. 8 in. square with a plinth 8-in. high and 5½ ft. square. The short columns falling on the upper half of the slope wall are carried directly on the slope wall lining which is thickened to form a stepped footing. The footings of the lowest of these columns are partly on natural and partly on filled ground; the footings of the remaining short columns are cut into the embankment section.

Groined arch roof

The span of the roof domes is 38 ft. 3 in. The spacing of the columns, center to center, being the span of the individual arch barrels, is 19 ft. 1½ in. The slabs have a thickness of 6 in. at crown, 7 in. at the quarter

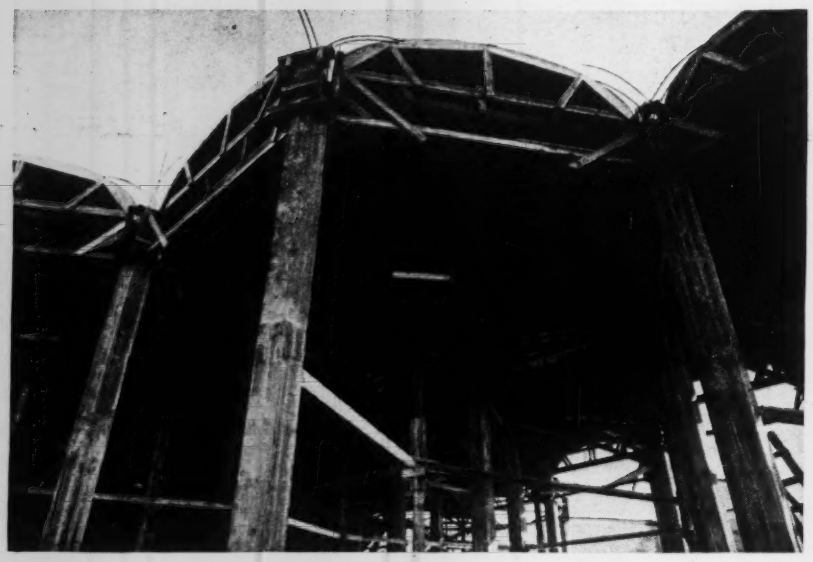


Fig. 2. Form work was relatively simple for this type of roof structure, each triangular unit having but one simple curved surface.

point, and 17 in. at springing over the column face. The mean vertical thickness is 8 in. including the column heads above springing lines. The rise of the intrados is 4 ft. 6 in. of which 1 ft. is below full water level. The diameter of the entire roof is 269 ft. No expansion joints were provided, reliance being had on the yielding of the arches and on light reinforcing steel to minimize contraction cracking. Although the roof slab stood exposed in part for some months only a few light contraction cracks opened before the earth cover was placed. They were at right angles to the arch barrels. The depth of cover above the top of the arches is 27 in. at the center of the tank and 15 in. at the rim, giving a 1-ft. slope for drainage.

Each column supports an equilateral triangle bounded by the center lines of the arches which spring from three of its six faces, the groins intersecting the center of its other three faces on the median lines of the triangle. The sides of these triangles along the crowns of the arch barrels are 35 ft. 1½ in. long.

No elaborate structural analysis of the roof was attempted. It seemed apparent, irrespective of arch or dome action, that the equilateral triangle supported by each column and forming an inverted umbrella-like structure springing upward and outward from the column, would be self-supporting. Failure of this triangle would be by splitting on the median lines from the angles of triangles to columns and the tension stresses on these surfaces are low. Two ⅝-in. reinforcing bars were placed across each of these groin lines near the angles of the triangle, the outer bars forming a ring with adjacent panels about each dome center. Other reinforcement also was light and was intended to resist temperature stresses and possible bending stress along the groin line.

Forms for the roof arches were built in triangular sections each forming one-half of the diamond constituting one barrel between four groins. Each panel was thus one-sixth of one of the hexagons. This type of roof involves no more expensive forming than that for the usual groins carried by columns on square spacing. In that case the arch form panels would be right triangles with 45-deg. angles; in this they were equilateral triangles. Some difficulty with forms arose from an original effort to build the panels for close fit. It was found more economical to allow 1-in. clearance between the edges of the panels which were bolted together through spacers and to cover the crack with metal.

Fill mixed and compacted

The excavated material consisted of 3 ft. of heavy black top soil, 6 ft. of clay, 2 ft. of sand clay mixtures, 4 ft. of soft caving sand and at the bottom 4 ft. of clean fine sand similar to beach sand. The entire bottom area and 84 of the 126 columns rested on this sand layer. The embankment was carefully constructed by mixing sand from the lower parts of the excavation with overlying clay in 6-in. layers.

The outer 3 ft. of the excavation for the slope wall was not excavated originally and the fill was built over this area with 3 ft. extra width. Just before placing the slope paving this 3 ft. was trimmed off of both natural soil and embankment. Settlement readings taken over many months around the rim of the concrete work and at points in the central portion of the reservoir indicated that portions of the reservoir resting on this fill behaved in the same manner as the interior which rested in the lower sand strata under loading and in elastic recovery on emptying the reservoir.

Lining and joints

The lining of the nearly flat bottom portion inside the ring joint at the foot of the slope lining consisted of an 8-in. slab resting directly on the sand except at the columns where this slab extended over the lower footings to the face of the plinths. This slab was lightly reinforced in both directions. At the rim point it lapped over an offset on the bottom edge of the slope wall slabs. The latter were 10 in. thick at the top and 12 in. at the bottom, increasing in the last 6 ft. from 12 in. to 16 in. at the bottom ring joint, with considerable reinforcing inclined in a radial direction. This reinforcing was placed near the upper or inner face of the slabs over the fill and for some distance below the top of the natural ground where it crossed over to the bottom of the slab for the lower portion. It was considered there might be a tendency of these slabs to bend outward and downward about the top of the unexcavated material as a fulcrum.

The relatively flat bottom lining was constructed with a joint around each plinth, an interior ring joint between the plinths of the six central columns and from this ring joint six radial joints on lines centered between columns extending to the ring joint at the foot of the slope wall. From the ring joint at the foot of the slope wall sixteen radial joints extended up the slope wall

dividing it into sectors about 50 ft. long. The slant height of the slope wall and curved section at its base was 43 ft. with a horizontal lip 2 ft. wide and 15 in. thick at the top.

Underdrain construction

No attempt was made to provide structural resistance in the lining slabs against possible uplift pressure. In order to protect against possibility of uplift in case of a rapid emptying of the reservoir, a complete system of tile underdrains was placed under the floor. It extends well out under the foot of the slope wall. The natural ground water level ranges from 7 to 11 ft. below the bottom of reservoir at the center. The underdrains consist of five radial drains leading into a manhole beneath the floor at the center of the tank. A sixth radial drain is a 10-in. bell-and-spigot concrete outlet line which also serves as a foundation drain.

These radial drains extend out to a point 11 ft. beyond the bottom of the slope wall with 4-in. stubs 10 ft. long at right angles on each side. Four 4-in. laterals lead off of each of these main radial drains, the outer pair also extending beyond the foot of the slope wall, with a cross header at the end. In this manner there is no point in the bottom area of the reservoir nor the lower half of the slope walls more than 13 ft. from a drain line. The joints in these underdrains were filled with a lean mortar made with coarse ungraded sand to give porosity for free entry of water and yet to prevent sifting of the fine foundation sand into the drains.

Before describing the method, difficulties and results obtained in waterproofing the reservoir, certain details should be mentioned. A 12-in. cast-iron outlet under the floor was provided from an open bell at the low point at the center of the reservoir. It leads out to double gate valves in a large outlet manhole situated outside the reservoir wall beyond the toe of the embankment. A vertical 10-in. cast-iron riser was connected into the main drain line carrying at full water level a circular overflow weir consisting of the 20-in. spigot and of a 10 x 20-in. reducer. The underdrain line also passes into the drainage manhole and connects through a 10-in. check valve with the main outlet drain below. The check valve is placed to close against back pressure from the drain below, which will occur from hydrostatic head upon opening the outlet valves on high levels in the reservoir.

The main outlet line and the underdrain and waste overflow line are connected at the manhole into a 15-in. reinforced concrete sewer with an outlet into a natural drainage course some 1,200 ft. from the junction manhole. An additional overflow wasteway at slightly higher level than the pipe outlet is provided over the rim of the reservoir with paved gutter down the back of the slope. This waste outlet is equipped with a hinged closure door, which is insect proof and will float clear under slight water pressure.

To protect the roof against possibility of air loading from partial vacuum created by rapid emptying, four 6-in. pipe air inlets are provided through the roof. These have a U turned down close to the level of the top of the fill and a concealed copper mosquito netting inside. The earth cover and connecting slopes were covered with top soil laid aside from the excavation and sodded with Bermuda grass. The irregular openings around and above the rim of the slope wall beneath the ends of the arch barrels are closed by a series of vertical reinforced concrete walls.

Waterproofing details

For waterproofing, reliance was placed on the lining slabs previously described. The contraction joints between the various slabs were all "cold joints" with a groove 3 in. deep cast in the second slab poured, tapered from $\frac{3}{4}$-in. width at the surface to $\frac{1}{2}$ in. at the bottom. The paving slabs were placed alternately, in order to minimize shrinkage opening at the joints. The specifications required that the reservoir be loaded with water as much as possible before closure of any of the joints in order to permit the slabs to settle to final bearing to the maximum possible extent. The contractor was to deliver the reservoir watertight within a tolerance of 30,000 gal. leakage in 24 hours. At this rate of loss a penalty of $3,500 was to be applied. This penalty was to be scaled down proportionately to nothing if the leakage was brought below 10,000 gal. per day.

On completion of the concrete work, including the lining, and placing of the earth roof load in the spring of 1937, water was admitted to a center depth of 24 ft., 10 ft. below full level. The leakage at this time with the joints entirely unsealed was 660,000 gal. per day, of which 255,000 gal. of clear water passed from the underdrain outlet. On account of difficulties which the city water department was experiencing in maintaining service, due to an increase in population served of 50 per cent within a period of a few months, it was necessary on completion of the above preliminary tests to suspend construction and the remaining work of sealing the joints.

The city took over and utilized the reservoir at levels corresponding to storage of 2 to 6 m.g. during the summer and early fall of 1937 in

Fig. 3. Each equilateral triangle of half arches is supported by a single column under the low point, right center foreground.

order to carry daily peaks beyond the capacity of the delivery main and other distribution storage in the city, notwithstanding augmentation of the delivery rate by installation of an additional booster pumping station on the supply main. At the end of October consumption of water had decreased to a point permitting the city to release the reservoir from service use. At this time the leakage at low levels, corresponding to 60 per cent capacity, had decreased to about 40 per cent of that observed on the initial filling, the joints having remained entirely unsealed.

Rigid caulking unsuccessful

Caulking of the joints around the column plinths and throughout the slightly sloping bottom slab and the ring joints on the 91-ft. radius was then completed under the original specifications and the radial joints in the slope wall, where movement was most feared, were partially caulked under the same specifications. The type of joint specified by the engineers, following their successful practice in other reservoirs on firmer soils, consisted of cement and oakum placed under the same specifications as used for cement joints for water mains.

This work was completed early in December during which month a further test was made with large reduction in the leakage noted up to levels 14 ft. and 4 ft. below full level. At these points relatively sharp increases in the leakage curve developed and at full level the leakage amounted to 525,000 gal. per day with considerable discharge of clear water from the underdrainage system. The reservoir was immediately emptied and inspection made. This revealed only slight opening of the slope wall joints but all the radial joints in the floor and the ring joint had opened. The joints around the column plinths were found uniformly sound with the exception of those around the central six columns which had opened as had the connecting ring joint between them.

Most of the joints which had opened were at some distance from columns and were found to have opened more in proportion to the distance from columns. These openings in the cement joint beads varied from 1/128 to 1/64 in. Considering the large length of joints this width

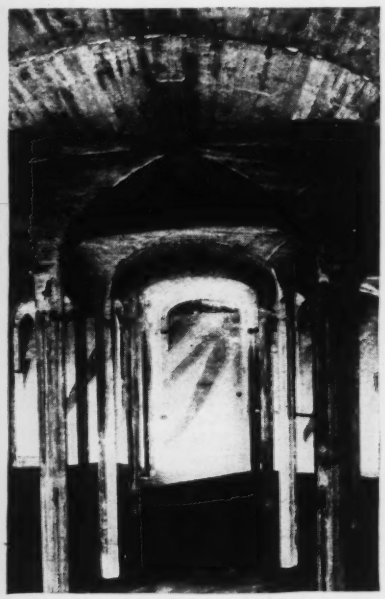


Fig. 4. Interior of reservoir after the roof and column forms had been stripped.

was ample to account for all or a considerable part of the leakage. It is believed that these joints opened at two stages of filling corresponding to the points of sharp increase in the leakage curve. Leakage observations as the filling progressed showed practically none up to a level with the base of the slope wall, gradually increasing for the next 16½ ft., the first sharp increase which was believed to represent the first point where the cement joints began opening.

These observed conditions lead to the belief that the bottom slab placed with vibrators on a 4-per cent slope was practically tight and that the slope wall slabs, which had been placed without top forms with a dry mix against a 1:1 slope, were responsible for possibly 40 per cent of the total leakage. It seemed apparent that the opening of the joints was due to flexure of the slabs on the elastic soils due to the warping of the slabs at the columns, where loads were slightly heavier. This belief was supported by the wider opening of the joints with increased distance from the columns.

Pitch and fabric seals

It was decided that the use of the rigid cement-and-oakum joints for the lining slabs on the elastic foundation soil was quite evidently a mistake. The cement and oakum in these joints were accordingly entirely removed with the exception of the plinth joints around columns, which were intact, where the upper bead only was removed. The joints and adjacent edges of slabs on each side were thoroughly cleaned and dried and the faces of the joint and the slab edges coated with a priming coat of special grade coal-tar pitch suitable for the maximum of penetration and adhesive value. Oakum was impregnated with hot coal tar at the job and the joints filled with this tarred oakum and caulked tight nearly to the top. In the case of the plinth joints the oakum was omitted and the priming coat applied over the adjacent slab edges and the upper surface of the originally placed cement and oakum.

After the priming coat and oakum were placed the joint was swabbed for a width of 18 to 24 in. with hot coal tar pitch, using every care to see that the previously applied coat was absolutely dry. A layer of waterproofing fabric cloth, 9 in. wide, was laid immediately in this first swabbing of tar extending along over the joint and lapping onto the adjacent slabs on either side. A second swabbing of the pitch and a second layer of fabric 12 in. wide was then placed followed by a third swabbing of pitch. The coal tar pitch was specially selected of a quality to obviate the tendency to flow down the joints in the 1:1 slope wall; the same material was used for the other nearly level joints.

Grout and bentonite

The remainder of the slab areas constituting nearly the entire area of lining was treated with three applications of neat cement grout containing a small amount of waterproofing admixture. These applications were applied also over the coated strips at the joints in order to minimize the possibility of taste from the tar in the water. Some three tons of bentonite powder was finally sprinkled over the whole area of the reservoir as a final precaution against any seeps which might remain unclosed.

Final test was made with no observable leakage up to an elevation of 11.6 ft. below full level. Slightly increasing leakage was observed from that point to full level at which the final leakage was observed as 7,400 gal. per day corresponding to

a drop of 7/32 in. in 24 hours. This loss included what evaporation occurred in the covered reservoir.

Costs

The extra cost for cleaning out the cement and oakum joints and for constructing the new joints including material, labor, equipment charges, construction supervision and 15 per cent profit to the contractor, amounted to $4,756. This covered the removal of the cement and oakum from the joints, the complete oakum and tar and fabric covering for 3,025 lin. ft. and the removal of the cement bead only with installation of the tar-and-fabric cover for 1,460 lin. ft. The cost of the cement washes and bentonite was borne by the contractor under his original contract.

The entire cost of the reservoir, drains, discharge lines connecting to the distribution system, the pump station serving it, the site, including space for additional reservoir, the cost of the extra work on joints as above described, with engineering and inspection was $177,732. $73,678 was a PWA grant and the remainder from waterworks funds.

The work was initiated under the administration of H. R. Giles, mayor and R. A. Thompson, water commissioner, and contracts let under the administration of his successor, John A. Mobley, water commissioner. The reservoir was completed under the administration of A. C. McCaughan, mayor, D. A. Segrest, water commissioner, and Joseph Mireur, finance commissioner. John W. Cunningham was superintendent of the waterworks throughout the undertaking. The reservoir was designed by The Terrell Bartlett Engineers, San Antonio, Tex., with A. M. Erskine of that firm as resident engineer. The contractors were Chamberlain & Strain of San Antonio.

Problems of a Disposal Plant

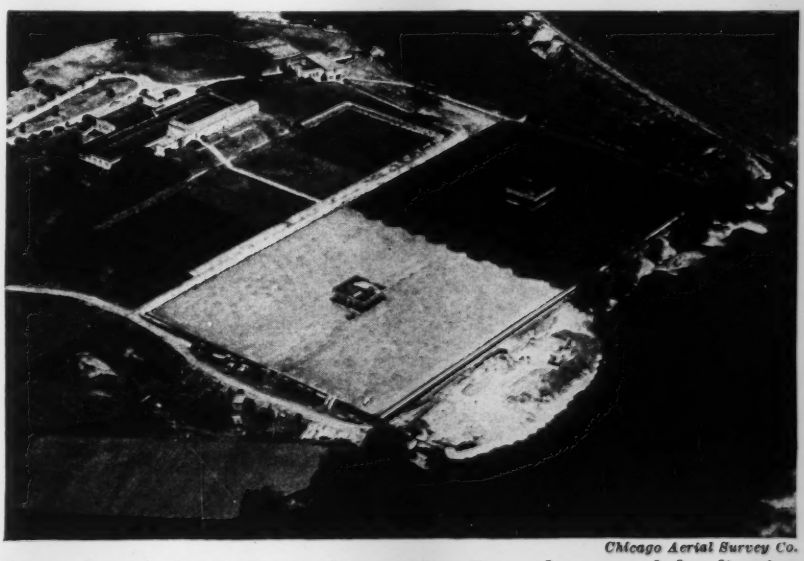
Far from being routine, operating activities at Aurora, Illinois, constantly challenged the ingenuity of the disposal plant's operating staff

INTERESTING details of performance and difficulties at the sewage treatment plant of the Aurora (Ill.) Sanitary District are given in the district's report for the calendar year, 1937. The volume of sewage treated at this trickling filter-separate sludge digestion plant was 2.3 million gallons. Sewage flow was about 12 per cent less than in 1936 and 1935, due to 12 per cent less rainfall (28.3 in. for 1937), but the strength was about 14 per cent higher in suspended solids. Of the total weight of sludge delivered to the digesters, 19½ per cent was from the new secondary settling basin, which was put in service on January 1, 1937. Under these conditions there was an increase in gas volumes and in operating difficulties.

Of 1,787 tons of solids in the raw sewage, 82.7 per cent were removed, leaving 17.3 per cent in finely divided solids discharged to the Fox River. Of the total removed, 1.1 per cent were screenings, 8 per cent grit; 1.5 per cent grease-scum; the remainder was finely divided solids, of which 51.9 per cent was removed by the clarifiers, 12.8 per cent by the filters, and 7.4 per cent by the secondary tanks. The 4,100 cu. ft. of screenings were burned in the incinerator. Grit totaled 7,800 cu. ft. of sand, cinders and street washings, averaged 61.1 and 35.8 lb. per cubic foot on the wet and dry basis respectively, and had a volatile matter content of 43 per cent. Grease and scum, amounting to 1,225 cu. ft. weighed 45 lb. per cubic foot dry.

The 1,556 tons (38,600 cu. yd.) of solids removed by the clarifiers represented a removal efficiency of 58.2 per cent and was an increase of 22.9 per cent over 1936, due mainly to heavy supernatant discharge in May and September, resulting from heavy plant loading. In addition, 191 tons of secondary sludge were discharged from the filters.

Among minor matters, an experiment was made in the fall by partly covering the plant lawns with sludge, with the idea that it might promote a more vigorous growth of grass which would choke or check the dandelion growth. About 800 cu. yd. of sludge was taken from the sludge pile for use on other lawns and local golf



Chicago Aerial Survey Co.

Sewage disposal plant at Aurora where trickling filter and separate sludge digestion treatment is employed. Plant capacity is 2.3 m.g.d.

links. A "Free Sludge" sign was put up, and some progress was made in interesting farmers to remove this material. Rodding the 4-in. pipe line conveying grease from the clarifiers being ineffective for removal of the hard caked grease, the pipe was cleaned with a motor-driven rotary device. Application of this scheme will be required only at intervals of two or three years.

Gas and power

Yield of gas was 18,079,100 cu. ft., an increase of 8.3 per cent. An increase from 0.97 cu. ft. per capita in 1936 to 1.01 per cent in 1937 was due partly to the secondary sludge additions. Volatile matter in the primary and secondary sludges averaged 66.6 and 58.2 per cent, respectively, with a weighted average of 65.2 per cent as delivered to the digesters. Owing to the lower gas content of the secondary sludge, and the charge from single-stage to two-stage operation on Sept. 1, the average gas yield per ton of dry material was 13,500 cu. ft., as against 16,600 cu. ft. from 1931 to 1936. At the calculated weight of 64.3 tons per 1,000 cu. ft., the gas yield represented 581 tons, or 159 tons per day.

Performance of the gas engines was highly satisfactory. Gasoline was used for 26 hours, but only for the sake of experience. The performance appeared equal to sewage-gas operation, except that costs were somewhat higher for the current generated. On the basis of net cost, the engines paid for themselves early in 1938, after two years of operation, and they have effected a marked saving in plant operating costs. With the present rate of gas production, installation of an additional engine-generator unit in 1938 or 1939 will further reduce the electric bills and the operating costs.

Two-stage digestion

The change in digester operation noted above consisted in converting the three single-stage digesters to a two-stage system. This was done because of the increasing sewage load and the further increase in solids collection from the new secondary settling basin. The work was done by plant labor at a cost of $1,412, and is expected to postpone for some time the costly construction of additional digesters. Laboratory studies preparatory to the change confirmed the anitcipated results. These experiments were made with three 5-gal. digesters, sludge from the bottom of the one first-stage digester being divided equally between the two second-stage digesters. No definite conclusion is yet possible, but tests indicate that first-stage gas will be richer than that from the second stage, and that the average will be higher in B.t.u. value than the gas produced by the single-stage process.

The problem of handling the increased load was intensified by the increase in gas release to 6.48 cu. ft. per day per sq. ft. of digester, as compared with 4.53 cu. ft. in 1936. Continued difficulty with short runs in the digesters led to adoption of the new system in order to handle the heavier loading with the existing equipment. This will extend the storage capacity of the digesters between cleaning periods and so reduce the sludge drying turnover. Three times in 1937 the quick filling of the digesters, with long periods of heavy sludge return in the supernatant, threatened to close down the plant. At one time, this caused 18 in. of sludge to accumulate in the clarifiers, with sludge beds not ready to be cleaned.

After a few days of operation on the new system, it was found that three zones were forming in the first-stage digester. The upper zone was an unusually heavy sludge of 12 to 15 per cent solids, possibly the flotation result of heavy gas discharge. For about 3 ft. at the bottom the percentage was only 3 per cent, while the central zone varied from a fairly clear supernatant to a sludge of 2 per cent solids. To reduce the load on the second-stage digesters and to retain the digestive activity as much as possible in the first stage (so as to extend the periods between cleanings in the second stage), it was considered desirable to change the method of transferring sludge continuously from the bottom of the first-stage digester. This was effected by means of a galvanized pipe 10 to 12 ft. long, having long slots on opposite sides in its upper 5 ft.

This pipe was telescoped through the emergency overflow pipe in the transfer box, being adjusted to such depth that the liquor from the center zone could be more or less selected as to sludge content. Sludge was prevented from rising in the transfer box from the regular pipe extending to the bottom by building up a head with the usual control rings. With this arrangement, the overflow from first stage discharged from the emergency overflow through the slots in the sliding pipe. Pumping is mainly from the center and thin zones, but about once a week there are one or two full pumpings taken directly from the bottom, so as to keep the bottom pipe clear of heavy sludge.

Filter troubles

Clogging of the filter nozzles occurred after an extensive and persistent flow of grease and oil, which material carried very little volatile matter and appeared to be of mineral-oil origin. Sluggishness in the operation of the nozzles led to examination, which revealed that the riser pipes were filled with a mass of small, firm and perfectly round balls, which when crushed proved to be of a gummy, greasy nature. Apparently this resulted from the character of the material and its movement up and down in the pipes by the fluctuating water column. Every nozzle had to be removed and cleaned, and bushels of these balls were blown out.

Rodding and flushing of the filter underdrains was done in 1931, at considerable expense. Since then, machines for cutting roots in sewers have been devised, and cleaning by some such device may be advisable in 1938 or 1939.

Operation of the new secondary settling basin was interrupted in May by a flood in the river, water being backed through the discharge pipe and even over the weirs of the basin to such extent as to admit fish. Removal of the fish from the basin was a problem, as cyanide treatment might affect the river, dynamite might endanger the structures, and lime might cause difficulties with the sludge pump. A heavy dose of copper sulphate (about twenty times the killing dose of 0.33 p.p.m.) was applied without success. Finally, the basin was kept out of service for several days, resulting in depletion of oxygen and consequent death of the fish, which gradually floated to the surface and were removed.

Walter E. Deuchler is engineer, and Walter A. Sperry is superintendent for the Aurora Sanitary District. The city has a population of 49,000, with combined system of sewerage.

Steel Gates Close Gaps in Dam

Concreting low gaps left in Grand Coulee spillway was made possible by 70-ton gates moved along the dam as work advanced

STEEL GATES of unusual size and design were built to effect closure of the gaps left temporarily in the spillway section of Grand Coulee Dam. Use of the gates made it possible to unwater the gaps, thus permitting concrete to be poured to a height above water level. After concrete placement, the gates could be removed for use in other gaps. Each of the gates, of which nine were made, was 52 ft. wide, 35 ft. high, and weighed about 70 tons. A primary purpose in their design was to facilitate easy handling in placing and removing them under a considerable head of water.

A seat and guide for each gate was provided by a steel frame built to fit around the lower portion of each gap. Eight gates and nine gate frames were built; the extra frame made it possible to transfer a gate from one gap to another without waiting for the frame to be moved. Each gate consists of nine horizontal girders about 3 ft. deep at the ends and 5 ft. deep for a distance of 15 ft. each way from the middle.

Steel girders were used along the sides and there was a steel plate cover on the downstream face. Along each vertical edge, on the downstream face, were mounted a row of 20 steel rollers, 16 in. in diameter, with faces 5⅛ in. wide. They ran on steel roller guides on the gate frames and made it possible to slide the gates in the guides while under water pressure. The lower part of each gate contained a closed chamber about 5 ft. high. Valves were put in for filling and draining the chamber and for passing water through the gate to equalize pressure on its up- and down-stream faces.

The gate frame consisted of a heavy, vertical H-column on either side of the 50-ft. gap, a box girder connecting the lower ends of these columns and a removable lattice truss

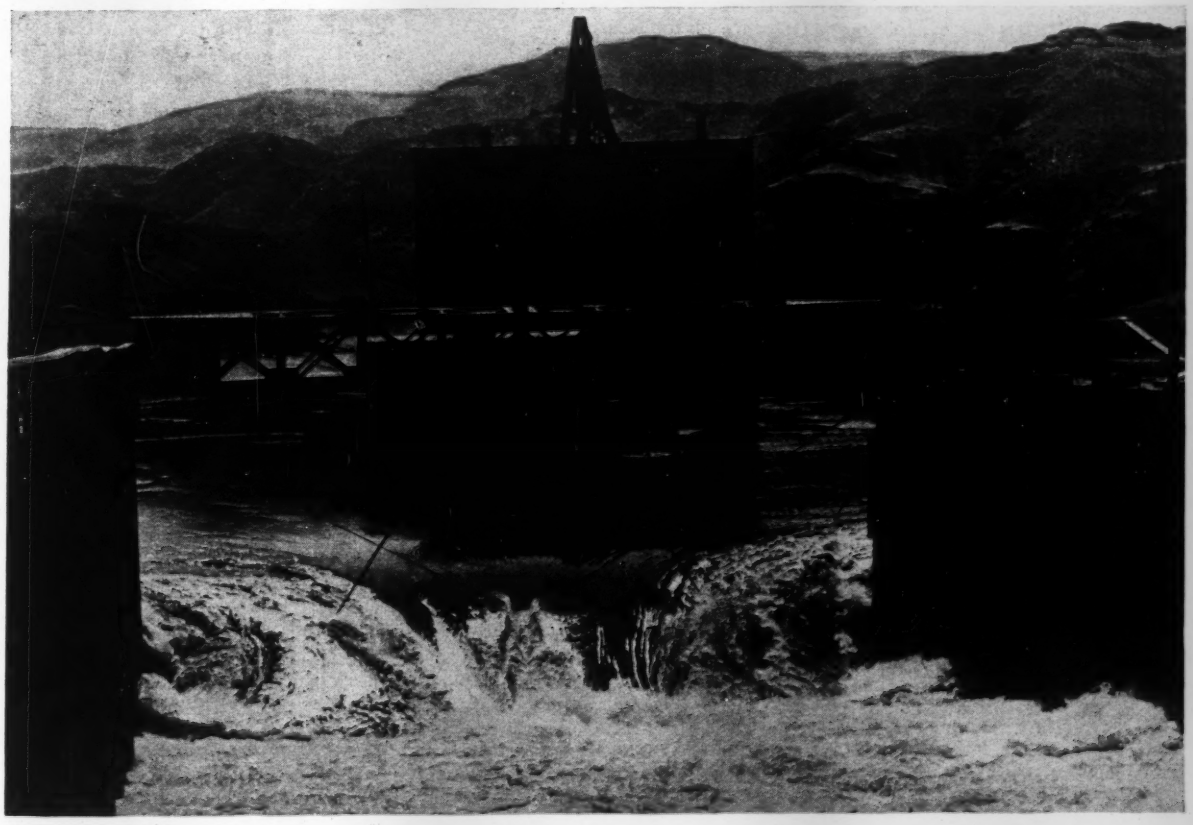


Fig. 1. One of the gaps prior to closure. Note lattice truss spacer in place on tops of guide frame. The A-frame of hoist barge appears above gate on transfer barge.

to be put on to the tops of the columns to hold them in place before the gate itself was put in. A dressed timber filler on one flange of the H-column made contact with the concrete and a projecting piece of ⅜-in. rubber belting clamped between the wood and the steel was forced by the water back of the gate into water-tight contact with the concrete.

The gate frame was suspended from brackets hooked over anchor bolts in the concrete, its weight and the force of the current entering the gap keeping it in place without additional fastening. After a frame was ready, the gate was set in it by a hoist barge carrying an A-frame and accurately controlled in position in front of the open gap by heavy cables anchored upstream. With the A-frame the gate was lifted from a transfer barge on which it had been brought up within convenient reach. The vertical row of rollers at the edges of the gate was then entered in the roller guides and the gate was lowered to rest upon the sill located at the bottom of the gate frame.

A water-tight joint between gate and frame was made by rubber gaskets between machined surface on the bottom of the gate and the top of the frame sill and by ⅜-in. belting strips attached to the outer vertical edges of the gate which are forced into contact with the frame by water pressure.

After a gate was in position and secured to the frame, the lattice truss joining the tops of the vertical columns was removed to make room for concrete forms for the upstream face of the dam. Seepage was pumped from the sump formed by the gate and the frame below the entrance to the gap. The gaps were sloped to drain downstream after water access was stopped by the gate. When concreting was finished water was admitted through the gate to equalize pressure on its two faces before the gate was removed by the A-frame on the hoist barge. Required hoisting capacity was reduced by using compressed air to force water out of the water chamber at the bottom of the gate.

The gates were designed by and are the property of Consolidated Builders, Inc., contractors for completion of the dam.

Hurricane Shelters in Florida

Concrete school houses on Florida Keys built to serve as community storm refuges

TWO UNIQUE BUILDINGS have recently been completed by the Works Progress Administration on the Florida Keys to provide shelter for the inhabitants against another such hurricane as devastated this area on Labor Day, 1935. Designed for normal use as schools, the buildings are equipped with kitchens and provided with water supply and sanitation facilities for emergency use during the hurricane season, which is in the Fall of the year.

Communication planned

Two-way radio service is planned for the buildings and a gasoline engine power plant is to be installed to keep radio and lights in operation. Cots and blankets are stored for 200 persons. One building is located at Tavernier on the lower end of Key

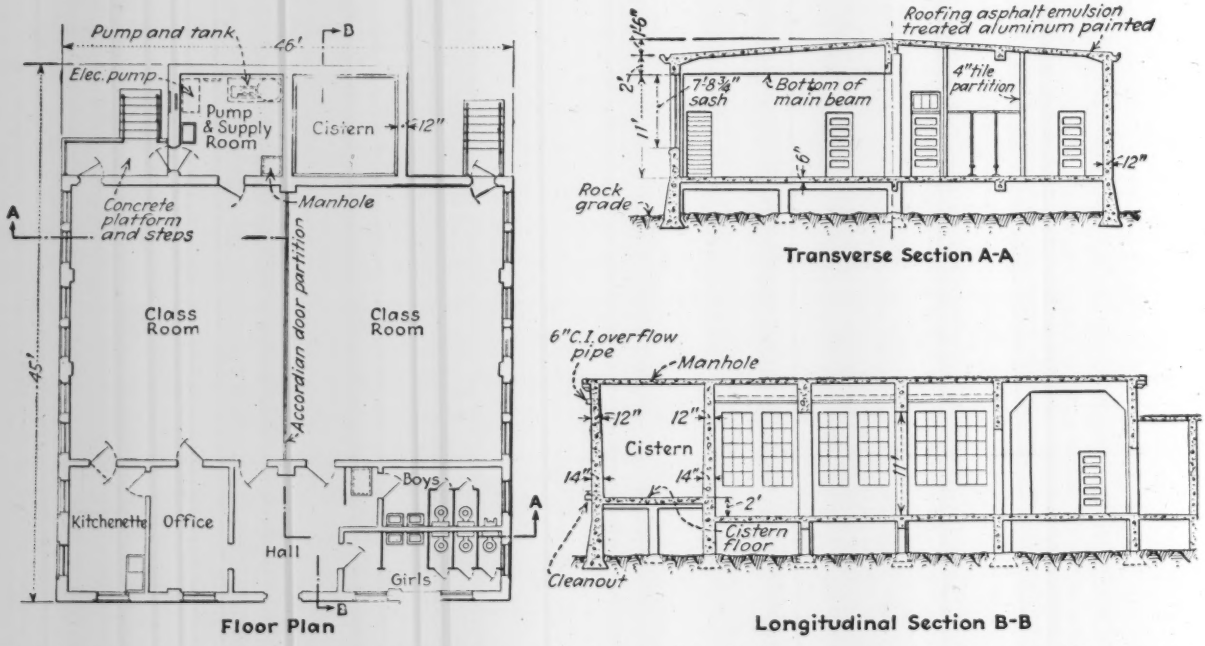


Fig. 1. Built of reinforced concrete and designed to resist the pounding of hurricane wind and debris, this Florida schoolhouse is equipped to serve as emergency living quarters for the nearby population.

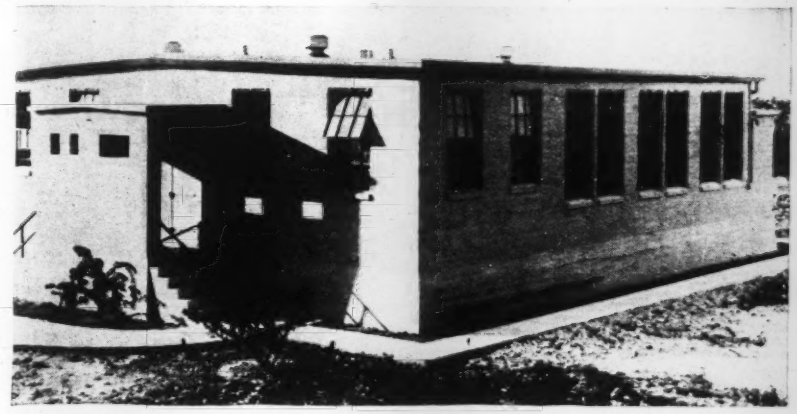


Fig. 2. Port in a storm is this school house on the Florida Keys designed to provide a community shelter in the hurricane season.

Largo and the other is on Upper Matecumbe Key.

As shown in the accompanying illustrations, the buildings are one-story in height and of reinforced concrete. Walls are 12 in. thick to resist flying objects as well as direct wind pressure. The 6-in. floor slab is supported 4 ft. above the rock surface of the keys and 14 ft. above mean low water so that doors are out of reach of wave wash. Floorbeams are carried on the walls and short intermediate columns, and both walls and columns rest in holes or trenches of keystone cross section that extend 1 to 2 ft. into the rock to resist uplift. The walls are continuous around the building so that water cannot get beneath the floor. Roof and walls are made monolithic. Steel framed windows equipped with heavy wire glass are used, and in addition there are wooden shutters 1 in. thick reinforced with steel angles and straps and assembled with bronze bolts to be put up in case of storm. A concrete wall, placed in front of the entrance, is designed to keep debris from striking the heavy timber doors.

Design loads

Live loads assumed in the design included 100 lb. per sq.ft. for the floor and the same for the roof, including wind. Wind and debris impact against the walls was assumed at 250 lb. per sq.ft. on any one bay and 150 lb. per sq.ft. over the entire wall area. A 700-lb. per sq.in. unit stress was used for the concrete (designed for a 3000-lb. ultimate strength) and 18,000 lb. per sq.in. for the steel reinforcing. Wall reinforcement, consisting of ½ in. diameter rods 6 in. on centers both ways, is placed 4 in. from the inside face of the wall. The vertical rods are bent down at the top and concreted into the 6-in. roof slab. A keyed construction joint is used between the wall and the floor.

Water supply

The cistern, whose superstructure is part of the building, has a capacity of 9000 gal. of water which is collected from the roof of the building in concrete gutters, monolithic with the roof. The buildings are reported to have cost $20,000 each.

Central Curbing Separates Traffic On New California Highway

RELOCATION of the highway over Altamont Pass in California, with greatly reduced grades and curvature, has afforded opportunity to install curbed separation between opposing traffic on a main route between the San Francisco Bay area and Central Valley points. The curbs are 8 in. high now but will be 6 in. high ultimately when another 2-in. pavement thickness is added. Their outer faces are 4 ft. apart (at pavement level) and the space between curbs is filled with gravel. Crossovers 16 ft. wide for use of maintenance equipment only, are put in every 1,500 ft. At intersecting roads and for farm crossings, the central curbing is discontinued for 30- to 40-ft. lengths and these gaps are plainly marked with "keep to the right" signs. The two traffic lanes next to the curbs are 12 ft. wide; those on the outside of the road are 10 ft. wide.

The old route was a bottleneck whose sharp curves and heavy grades slowed up passenger autos behind slow moving trucks. Last year the average summer traffic over the old route was 9,000 vehicles per day, of which 20 per cent was trucks.

Fig. 1. Central separation for opposing traffic on the new Altamont Pass highway.

Imperial Dam and Desilting Works

Dedication marks completion of unique irrigation structure on Colorado River, now ready to serve All-American and Gila Canals

DOWN the Colorado River below Grand Canyon, three great structures now impede the flow of this famous stream, each to exact, in turn, its measure of usefulness from the water which heretofore was largely wasted to the sea. First, at Boulder Dam the water turns turbines to produce electric power. South, 155 miles, Parker Dam diverts 1,100,000 acre-ft. a year for municipal water supplies. And finally, 150 miles below Parker, the water is subjected to another partitioning at Imperial Dam, this time for irrigation—12,000 sec.-ft. to California's Imperial Valley and 2,000 to Arizona's Gila Valley in the initial installations. What water is left goes to Mexico to be used or discharged into the Gulf of California.

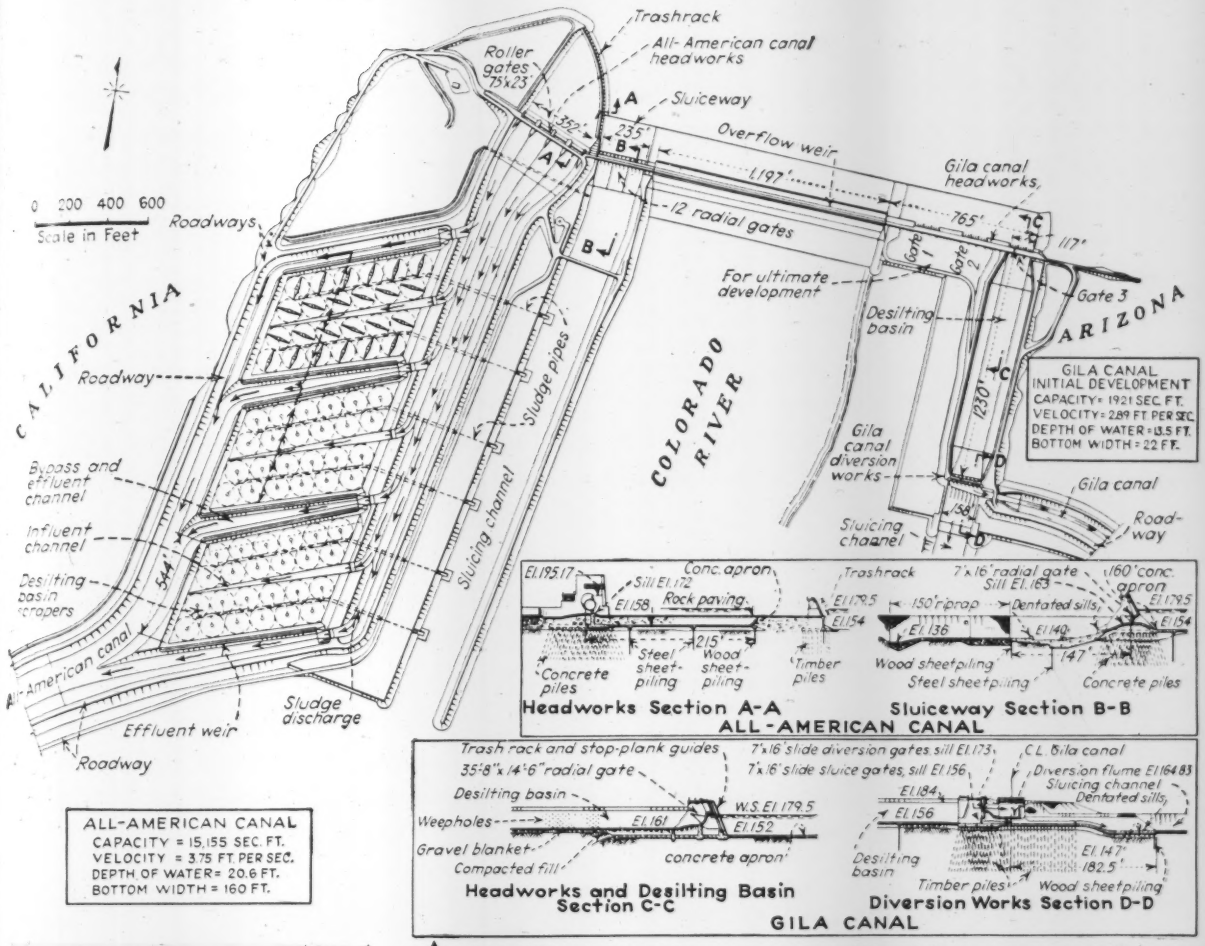
The dedication of Imperial Dam in October marked another milestone, and nearly the final one, in the great construction program ushered in a decade ago when Congress passed the Boulder Canyon Act.

Imperial Dam, presented on these pages in a picture story, is one of the world's unique engineering structures, as much a water purification plant as a dam. Designed by the U. S. Bureau of Reclamation, it is ready to go into service as soon as the All-American and Gila Canals are complete.

In the meantime, silt-laden river water is being passed through the dam sluice gates and around the desilting works into the first completed section of the All-American Canal, which will be used as a 20-mile long settling basin until the silt has provided the necessary watertight lining in the sandy bottom and sides. A similar seasoning operation will be required on the Gila Canal before its desilting basin is put in service.

Previous articles, which give engineering and construction particulars of the project, have been published in *ENR*, Oct. 17, 1935, p. 538 and Jan 21, 1937, p. 92.

MECHANICAL desilting works for the All-American Canal (below) dominate the California side of Imperial Dam. Water passed through the dam enters the basins through slots in the wedge-shaped channels and, after 21 minutes detention, spills over into the effluent channels minus 80 per cent of its silt load, or 70,000 tons from each day's flow of 7¾ billion gal.

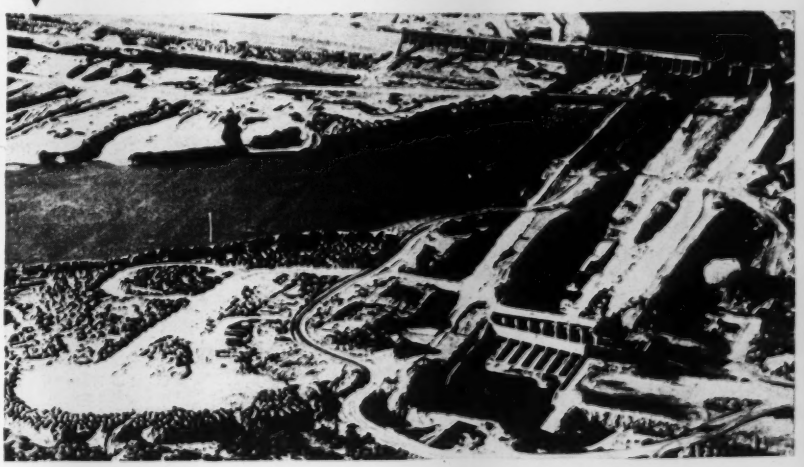


ALL-AMERICAN CANAL
CAPACITY = 15,155 SEC. FT.
VELOCITY = 3.75 FT. PER SEC.
DEPTH OF WATER = 20.6 FT.
BOTTOM WIDTH = 160 FT.

IMPERIAL DAM is large in extent and varied in character. The canal headworks on either bank are separated by a 1200-ft. ogee concrete overflow weir 26 ft. high and of hollow construction. Revolving scrapers, 125 ft. in diameter, remove silt from All-American Canal water, while Gila Canal water is clarified by straight sedimentation. The whole plant is supported on piles.

OVERSHADOWED in novelty by its mechanical neighbor across the river, the gravity desilting works for the Gila Canal on the Arizona side are nevertheless of major size. Two tiers of gates in the outlet works permit clear water on top to be diverted into the canal or the sludge on the bottom to be sluiced into the river through a channel not yet excavated.

TRIAL RUN, or for dedication purposes only. Since the ceremony on Oct. 18, raw water has flowed around and not (as here shown) through the desilting works into the All-American Canal. Only after enough silt has been deposited in the canal to make it watertight will the desilting works be put into operation.



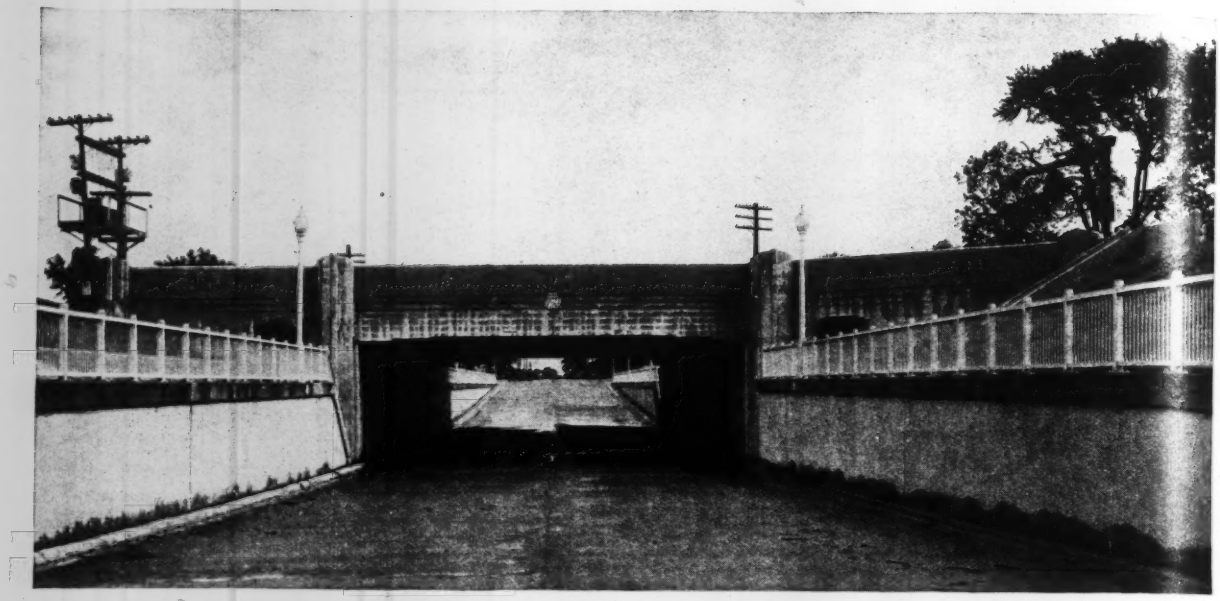


FIG. 1. RAILROAD UNDERPASS WHICH FLOODS WILL PROBABLY FILL ONCE EVERY FIVE YEARS

Underpass Built In A Flood Plain

LaMotte Grover

Assistant Professor of Applied Mechanics, Kansas State College. Formerly Office Bridge Engineer, Kansas State Highway Commission

Grade separation structure at Lawrence, Kans., waterproofed and designed to withstand uplift, and equipped with automatic pumping system

A TYPICAL example of the feasibility of constructing a subway grade separation between a four-lane arterial highway and a main line railway within a stone's throw of a major stream is afforded by an improvement opened to traffic recently at Lawrence, Kans. The subway, 900 ft. long, eliminates a hazard to safe driving that had become continuously more serious because of sharp turns with short sight distance that had to be negotiated by traffic on routes US 40 and US 59, in a short stretch between the Kaw River bridge and a grade crossing of the Union Pacific R.R. tracks. At this location the highway also serves as an important shuttle connection for the interchange of traffic between routes US 24 and K 32 on the north side of the railroad and river, and K10 and the main part of the city of Lawrence, on the south side.

With the adjacent north approach of the Kaw River bridge on a descending grade toward the railroad and only some 1000 ft. distant from the nearest of the three tracks involved at the chosen site, a viaduct grade separation appeared awkward from the start, especially in view of the difficulty of providing a safe means of access to the nearby Union Pacific passenger station. Added to this, there was the fear that a viaduct structure would appear unsightly in the face of existing improvements and the surrounding flat topography.

At the same time a very formidable objection to a subway was interposed on the grounds that such a crossing would be blocked by a high water stage of El. 100, which perhaps would occur as often as once in 5 years despite any practical system of local dikes; at least four times on record, the Kaw flood stage has topped the railroad tracks at El. 108. It finally developed, however, that the tentative Army Engineer plan for flood protection contemplated enclosing all of North Lawrence, including the site of the proposed grade separation, by dikes, and that the highway approach on the north could be carried up over the dikes to insure a high-water crossing under any circumstances. Accordingly a subway improvement was designed that would provide for uninterrupted traffic except when the water is above El. 100.

Designed for hydrostatic uplift

The adoption of a subway necessitated enclosing the entire depressed roadway area within watertight walls and floor up to El. 100, or 10¾ ft. above the lowest point in the highway grade. (Roadway grades of 3.3 and 3.5 per cent on south and north respectively are connected by a 400-ft. vertical curve).

Available data and construction experience would have led one to believe that a substantial increase in

groundwater elevation was not to be expected during floodtimes. Nevertheless, it was decided to take no chances, and enough weight was provided in the structure to counterbalance an assumed full hydrostatic pressure up to El. 98. The side walls and floor were designed at all sections to withstand an equivalent hydrostatic pressure of 100 lb. per cu.ft., using increased stresses of 1700 lb. per sq.in. in the concrete and 30,000 lb. per sq.in. in the reinforcing steel. That the anticipation of a change in the apparent permeability of the soil was a wise one was borne out by the effectiveness of the well-point system used during construction. (*ENR* June 10, 1937, p. 877).

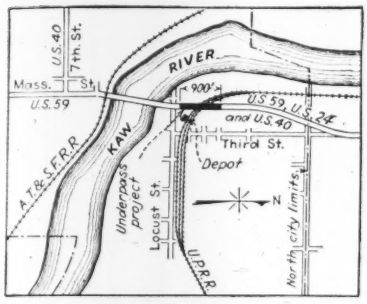


Fig. 2. Location of Lawrence underpass on important highway routes and main street of the town.

The first study for counteracting against buoyancy entirely by means of the weight of the concrete, resulted in a maximum thickness of base slab of 5½ ft., which was so excessive as to make the cost prohibitive. A source of very sandy clay borrow was then located in the vicinity for backfill between the base slab and the wearing surface of the highway that would provide a compacted density under rolling of at least 115 lb. per cu.ft., which was enough to permit a very great reduction in the weight of concrete that would not be offset by increased volume of displacement and an accompanying increase in buoyancy forces. The final maximum depth of base slab was only 18 in. A density considerably in excess of the assumed 115 lb. per cu.ft. was actually realized during construction. A typical cross section through the retaining walls, base slab, backfill and pavement slab is shown in Fig. 3.

Such a design called for an adequate system of underdrains to insure that the backfill under the pavement would not become saturated with surface water. This system consists of 4-in. porous clay tile laid under the pavement in a herringbone pattern, the trenches for the tile being filled with crushed stone. Furthermore, the entire outside area of the walls and base slab are protected with a three-ply membrane waterproofing which, in the case of the base slab, is laid on a 2-in. seal course of concrete provided expressly to give the membrane a uniform support.

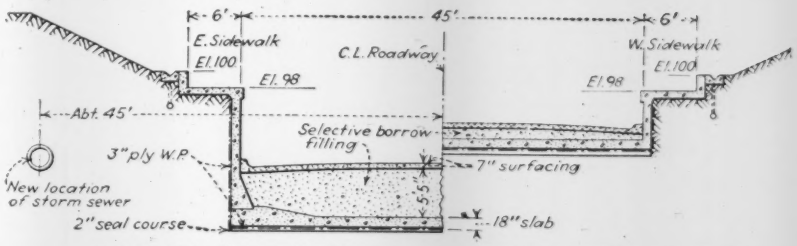


Fig. 3. Designed to resist hydrostatic uplift by its weight, subway box is ballasted by earthfill beneath its roadway.

A length of perforated corrugated metal pipe with porous backfill around it is placed behind each of the abutments of the railway bridge. These drains as well as drains from the walkways and the pedestrian tunnels under the bridge will discharge into an adjacent storm sewer with check valves at their outlets.

Drainage from the roadway area, including local run-off draining into side streets, is intercepted by storm-water inlets under or near the railway bridge, and carried to the wet well of a pumping plant, located about 20 ft. east of the east abutment of the railroad bridge. The water is discharged by the pumps into a catch basin built in an existing 30 in. storm sewer that was relocated and reconstructed of reinforced concrete culvert pipe in the immediate vicinity of the site.

Pumping system

The pumping system is composed of two 8-in., vertical, enclosed shaft submerged-type, heavy-duty, centrifugal pumps, delivering 1500 g.p.m. under a 27-ft. head, and operating at 720 r.p.m., driven by weatherproof, hollow-shaft, ball-bearing, 220-volt, 3-phase, 60-cycle, normal-torque, low-starting-current motors, with the thrust load carried on the top motor bearing, insuring proper alignment between the pumps and motors and ease of adjusting the impeller after installation. The motors were specified to be 15 hp., the complete pumping units to be of such a design that the motors would not overload under any possible condition of head.

Two pumping units afforded an especially significant advantage over one larger one in this case, by throwing a smaller starting load on the line, aside from the advantage of always having a unit in reserve. Each pumping unit is suspended individually from the floor of the motor room, resulting in a rather long drive shaft the casing for which had to be very well braced.

Although several reliable manufacturers strongly recommended enclosed pump impellers to eliminate the possibility of small trash or cloth catching between an open impeller and the volute, the type of impeller was not definitely specified, in view of the fact that maintenance records carefully kept over a period of fourteen years by a municipality operating some thirty automatic pumping installations for subway drainage indicated that non-clogging sewage-type pumps are not necessary. The pumps as furnished, however, are equipped with two-port enclosed type impellers, guaranteed to pass a 5-in. sphere, the shut-off pressure being 42 ft., which is enough to enable the pumps to build up a high pressure for throwing out obstructions.

The lower pump bearings are long and carry sufficient grease to run for a considerable period of time. The intermediate and upper bearings are oiled by solenoid-operated lubricator.

Automatic control

The control equipment is arranged so that the second pump is started automatically if the capacity of one pump is not sufficient to hold the

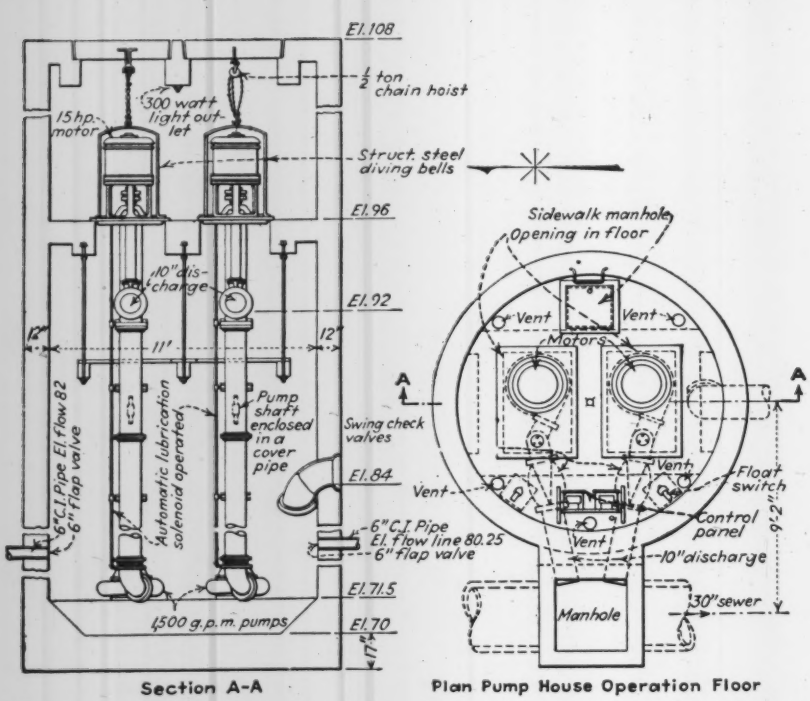


Fig. 4. Pump pit located at one side of project accommodates equipment that will automatically lift drainage water into storm sewer.

sump level below a predetermined elevation; both pumps then operate until the water is reduced to shut-off level. A setting is provided by means of a master switch and push buttons to permit the operation of the motors independently of the switches that are actuated by the level of the water in the sump or wet well.

Mechanical alternators with either float switches or Mercoid switches were permitted, but the control equipment as furnished is completely electric, consisting of a group of magnetic switches with fused disconnecting switches, relays, push buttons, starters and master switch, together with Mercoid control switches in lieu of float switches.

Pump house below highwater

Quite a thorough study and discussion of existing subway pumping installations revealed a general concensus of opinion that a pump house exposed to view is a serious handicap to esthetic construction, however pleasing its individual appearance may be. It was decided, therefore, to construct the entire pump house below ground. With the roof of the motor room flush with the ground and close to extreme high water elevation, provision had to be made for the protection of the electrical equipment.

Separate vent pipes are provided to ventilate the wet well and motor room and prevent excessive condensation on the equipment. The floor of the motor room is made watertight with gaskets around the openings under the motor bases, and a watertight hatch door leading to the wet well is provided to keep the water out except under the most extreme flood conditions (about once in 25 years). The motors are provided with special damp-proof insulation, and steel diving-bell covers are hung over them to be lowered in case the water is so high as to enter through the air vents and around the roof openings; under such conditions traffic could not reach the subway, and the motors could, therefore, be shut off.

To provide for getting the pumping equipment back in service after such an emergency, when both the wet well and the motor room might remain filled with water, a 14-in. bypass, with a gate valve operated from the roof of the motor room, was installed to let the water flow by gravity from the wet well into the storm sewer as flood waters subside. A second valve operates to let the water out of the motor room, so that operators can raise the diving bells and prepare to start the motors.

In addition to a covered manhole in the roof of the motor room, a removable slab of concrete is constructed over each pump unit so that the units may be removed for repairs. Such provisions eliminate the necessity of constructing a separate dry well to house the pumps, which is quite expensive.

Pumping capacity

To determine the capacity that should be provided by the pumping system, total accumulated rainfall was plotted against time for one storm that could be considered as typical of an 8-yr. frequency, and also for a number of other outstanding rainstorms of different time-intensity characteristics as well as for a U.S. Department of Agriculture 5-yr. frequency storm. The drainage area being small and a large per cent of it paved, a 100 per cent run-off was assumed. On each of these curve sheets lines could then be plotted to represent the cumulative volume of water that would be pumped out with pumping systems of various capacities, enabling one to determine for any proposed capacity, the volume of water that would be ponded in the subway area at any time during the storm, and the maximum depth and length of the pool of water as well as the time interval during which various conditions could be anticipated. The wet-well storage capacity proved to be an almost negligible factor in the computations.

It was found that during the typical 8-hr. frequency storm, lasting 1¾ hr., the roadway would be flooded for about 50 min. if two 1500-g.p.m. pumps were provided, and that the pool of water would be 5 in. deep and 134 ft. long. The U.S.D.A. 5-yr. frequency curve, which gives a good indication of what would happen in case of a short rain of very high intensity, indicated flooding for only 18 min. with a maximum pool 3 in. deep and 100 ft. long. The time intensity characteristics of storms of greater frequency were such that no flooding of the roadway would occur. In case of an extreme flood occurring before general flood protection works are constructed for the community, and the subway area becoming completely flooded, the two 1500-g.p.m. pumps will unwater the area within 8¾ hr. after the flood

waters have subsided to El. 100 so that inflow ceases.

The stormwater outlet is provided with a flood gate, mainly for the protection of the drainage district which is a joint user. The subway pumps will deliver full capacity against any head up to the flood elevation when water begins to overflow into the area.

Structural details

The railway bridge has a ballasted deck carried by a ⅝-in. wrought iron deck plate, resting on 36-in. wide-flange beams ranging in size from 150 lb. to 340 lb., depending upon their location with respect to the tracks, which are on a curve intersecting the centerline of the highway at 41 deg.-46 min. There are 57 beams on 2½-ft. centers, spanning 48 ft. 4 in. center to center of bearings, at right angles with the centerline of the highway. The beams were shop riveted together in sets of three for shipment and erection. The design was controlled by maximum allowable live load deflection. A very shallow deck, 4 ft. 9 in. in depth from base of rail to low steel, was dictated because of the very great cost of counterbalancing the increased buoyancy that would have resulted from lowering the highway grade. The supporting superstructure of the bridge is flanked by concrete fascia girders on either side over the portals of the highway opening. These girders are extended up to form parapets along the railway tracks, and they are designed to harmonize with the abutment and retaining walls.

The clear width of subway roadway is 47 ft. Walkways 5 ft. wide and elevated above the roadway surface are constructed monolithic with the retaining walls on both sides throughout the project, pedestrian underpasses being likewise provided on either side of the roadway at the railroad bridge. A comparatively light weight, open-type welded steel handrail is used along the sidewalks to avoid drifting of snow and collection of débris, there being no opportunity for traffic to collide with these railings.

The contract price for the project was about $267,000 exclusive of the cost of the highway pavement, the right-of-way, and items of work performed by the railroad company, which included a shoofly and temporary trestle bridge over the excavation for the highway, as well as the relocation or reconstruction of various railway facilities. With all construction costs added in, the project amounts to about a $300,000 improvement, involving the use of 384 tons of structural steel, 5750 cu.yd. of concrete, 293 tons of reinforcing steel, 112 tons of wrought iron deck plates and 30,000 cu.yd. of excavation. The cost of the drainage and pumping system, including underdrains for the roadway as well as the reconstruction of 1110 ft. of the 30-in. storm sewer, amounted to nearly $30,000.

The plans were prepared under the direction of Glen H. Trout, bridge engineer for the Union Pacific System, the details being worked out jointly between the railroad company and the state highway commission, for whom the writer was office bridge engineer at the time. Tom W. Oliver of the state highway bridge department made the hydraulic studies for the design of the drainage and pumping system, checked the subway design and made valuable suggestions for effecting economy.

The project was constructed by the Kansas City Bridge Co. for whom C. W. Cubbage was superintendent. J. S. Davis succeeded R. G. Porter as resident engineer for the state highway commission on the project when the latter was promoted to division construction engineer.

Chicago Park System Needs

CONSOLIDATION in 1934 of all Chicago's 22 park districts under the management of a single organization, the Chicago Park District (composed of five commissioners), eliminated the numerous large and small independent districts, with their 110 commissioners, complicated tax and finance systems, uncertain accounting methods and general irresponsibility. But the consolidation involved the new organization in serious physical and financial difficulties as the district had to assume charge of much uncompleted work and maintenance, as well as to assume liabilities aggregating more than $127,000,000. Because of these difficulties the commissioners, in 1937, appointed a citizens' committee to investigate the operations of the district, especially as to whether the objects of the consolidation had been fulfilled. A condensed summary of the committee's report is given below:

The city is receiving more and better returns from park expenditures than it did under the old system. In less than four years the district has brought order out of chaos and has coordinated park activities into a single and efficient operating unit. It has also reduced the debt or liabilities from $127,000,000 to $103,400,000, which may be paid off at the rate of $5,000,000 annually. Annual expenses are now about $7,855,000, as against an estimated amount of $11,000,000 under the old system.

Park costs are higher than in some other large cities, because the parks are used more intensively, while the district bears expenditures for policing, lighting and maintenance of boulevards which are not ordinarily included in park budgets.

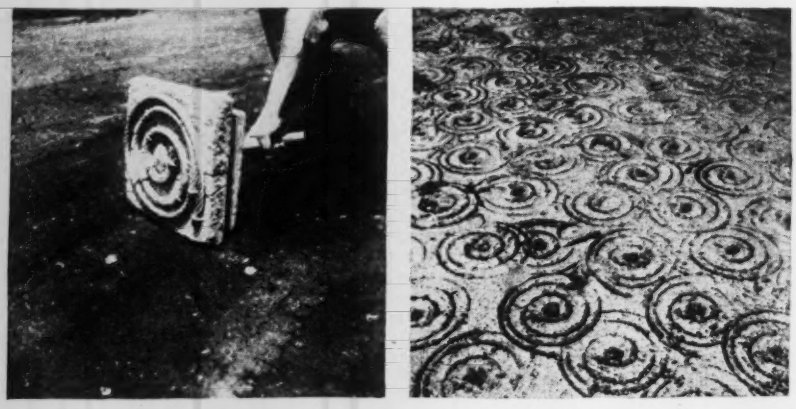
At the accepted ratio of 10 acres of park area per 1,000 population, Chicago should have 36,000 acres in parks and playgrounds (instead of the present 6,000 acres), according to the committee, and of this area about 10,000 acres should be of the small park and playground type. Thus the present area provides only 16 per cent of what is desirable.

It is recommended by the committee that a comprehensive plan for the acquisition, design and development of such facilities be prepared, and that the establishment of small parks and children's playgrounds be regarded as of the first importance. Many available sites are in sections where the initial cost would be low. Consideration should also be given to the possibility of obtaining sites through tax lien foreclosure.

Of the city's 35 miles of frontage on Lake Michigan, 28 miles are in parks, so that shore protection is a serious problem for the district. Protective works have cost $12,500,000, or an average of $450,000 per mile, exclusive of the cost of filled or reclaimed land, which averages $50,000 per acre. Only four of the 28 miles have sand beaches, but a more extensive use of such beaches is recommended, as a type of frontage that has proved effective for protection.

FROM FIELD AND OFFICE

Working problems and time-saving methods . . . Edited by Charles S. Hill



Concrete roughener and the work it does.

A Concrete Roughener

Where bituminous or sheet asphalt paving is to be laid on a concrete base, it is desirable to provide a roughened surface which will give greater bond. An easy way to roughen a newly laid concrete surface has been devised by Albert Greiner, concrete finisher in the street department at Altoona, Pa. He uses a 12 in. square tamping block on the bottom of which is fastened a three-turn spiral of wire cable, ½-in. in diameter. In the accompanying illustration the tamper is shown at the left and the type of surface produced by it is shown at the right. The tamping is done a short time after the concrete is poured.

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A Slide Rule for Yardages

FRANK J. AMADOR, JR.
International Boundary Commission, Las Cruces, New Mex.

A slide rule for yardage calculations can easily be made in a few hours and will prove a great time-saver in the engineering office. Computations of levee fills or channel excavation yardage can be performed in about one-third the usual time. In one setting of the slide the volume of a fill or cut of given end heights is read directly from the scale with an accuracy of 0.1 per cent. No indicator is required. If desired, an additional scale can be made which will permit the use of this calculator for sections whose base is not level. This added scale also gives the end areas corresponding to each height.

A desk model of this rule can economically and easily be made from a 2x4 in. board about 2 ft. long as shown. The slide is made of ¼ in. plywood, 1 in. wide and fits into a groove in the 2x4 in. body of the rule. The scales are made on drawing paper, cemented to the wood and protected by celluloid covering.

The rule consists of five scales located as shown. The volume per station scale is uniform. The scales for station A fill or cut and the station B fill are identical and can be made simultaneously to save time. The spacing for these scales is given by the relation:

$$I_h = \frac{h\,(t \times h\,\frac{S_1 + S_2)}{2}}{0.54 \times K}$$

where I_h = distance in inches from the zero point.
h = height of fill (or cut) ft.
t = top or (bottom) width of levee (or channel), ft.
S_1 = and S_2 = side slopes, ratio of run to rise.
K = scale selected for volume per station scale, cu.yd. per inch.

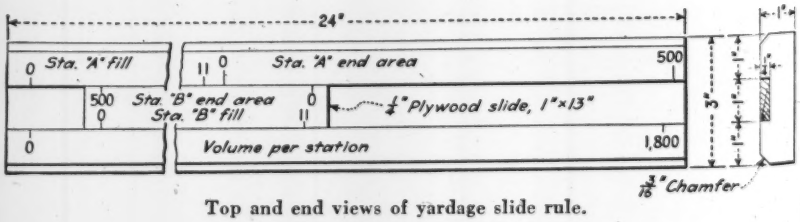
The station A end area and the station B end area scales are used when the ground is not level and the volume is computed in terms of the end areas rather than heights of fills or cuts. The spacing is determined by the relation:

$$I_A = \frac{A}{0.54\ K}$$

where I_A = distance from the zero mark, inches.
A = cross-section area of levee, sq.ft.

Care must be taken in locating the station A end area scale with respect to the volume scale. The zero point of the area scale should be directly opposite the point corresponding to the volume of a 100-ft. section having an average area of one-half the maximum value of the end area scale. The station B end area scale is a reversed scale placed on the upper portion of the slide with its maximum value in line with the zero point of the station B fill scale.

To find the volume of a levee (or channel) 100 ft. long with end heights A and B, set the left index of the slide on fill A and read the answer on the volume scale opposite fill B. If the area of fill is desired, it is found opposite the right index of the slide on the station A end area scale. If the ground is not level and the end areas are known, set the slide so that area B is opposite area A on the respective scales and the answer appears on the volume scale opposite the left index of the station B fill scale.



Top and end views of yardage slide rule.

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Marsh Hay Seals Pool Bottom

S. G. THOMPSON
American Shore and Beach Preservation Association, Newark, N. J.

Using salt marsh hay to prevent sand loss through an original muck bottom engineers for the Works Progress Administration and the

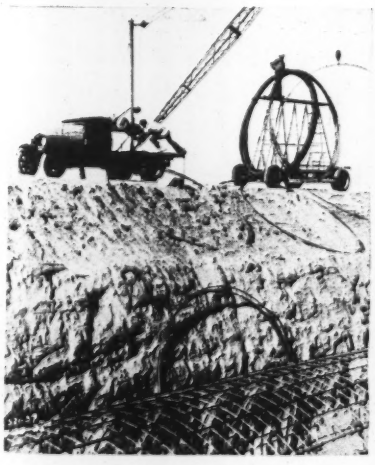
Camden County, New Jersey, Park Commission have created a 1,200,000-gal. pool and a sand beach in Roberts Park, Collingswood, out of what formerly was a dangerous "old swimming hole". While use of hay over muck has been common for years in the construction of roads this is believed to be the first time the principle has been applied to swimming pools. Tests made at regular intervals since a 12-in. mat of hay was spread and covered with sand at Roberts Park in the spring of 1937 show a negligible sand loss. What sand has been lost, the engineers say, has been carried over the weirs or taken from the pool on the feet or in the bathing suits of swimmers. The hay was not allowed to dry after being cut, but was spread immediately. So long as it remains under water it does not rot, but clings together in a solid mass, forming a pliable but tightly-meshed foundation impervious to sand.

Plans for the hay bottom were perfected by John H. Osler, engineer for the Camden County Park Commission, and engineers of the division of operations of the W.P.A.

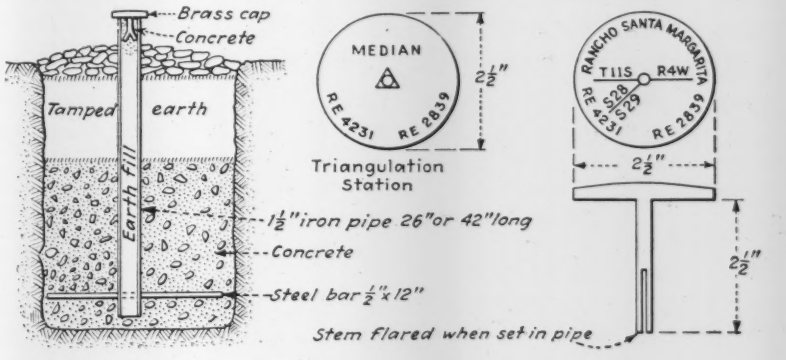
Rig for Delivering Steel Rings

A home-made rig was used for delivering reinforcing steel (made up in rings) to the trench in cut-and-cover sections of the Colorado River Aqueduct and is shown by the accompanying illustration. It was convenient to bend the steel for circular reinforcing in a nearby yard and to deliver it to the trench in the form of rings ready for assembly into the reinforcing cage that was built in place ahead of form- and concrete crews. After some experimenting with methods, the rig shown was developed and was used with great success. Essentially the outfit consists of a light motor truck equipped with a steel mast and derrick boom, towing a trailer fitted up with a rack for the steel rings.

The rig was developed by the Griffith Construction Co., which built some 12.9 mi. of the aqueduct.



Rig for delivering rings into aqueduct trench.



A REAL SURVEY MONUMENT

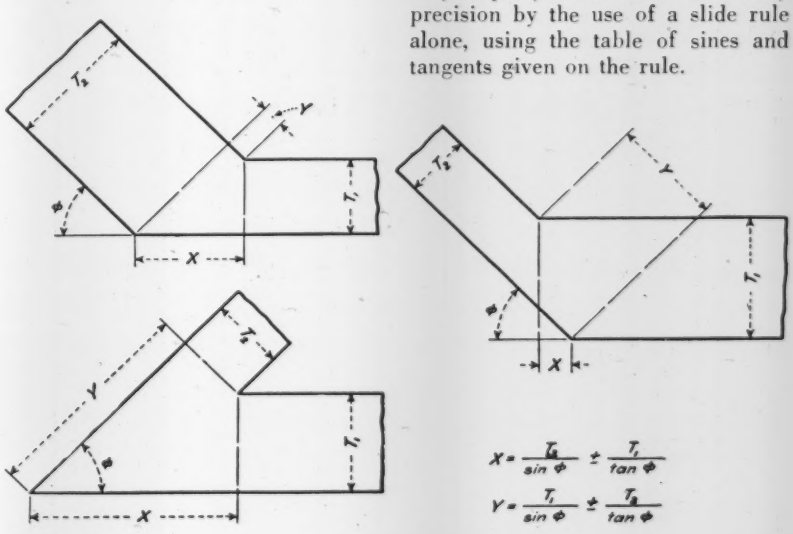
Monuments that surveyors set to mark land boundaries are often, for the pride of engineering, better left undescribed. Here is a monument however that was used in surveying the great Santa Margarita Rancho in California and that needs no apology for lack of permanence and clarity. It is illustrated in a recent news bulletin which is issued by the California Board of Registration for Civil Engineers.

Solving for X and Y

W. C. Goodwin
Truscon Steel Co., Boston

In Smoley's Parallel Tables of Logarithms and Squares is a method of solving for X and Y when T_1, T_2 and ϕ are known. Smoley uses two formulas for each of three cases, and makes use of four trigonemetrical functions, namely tangent, cotangent, secant and cosecant. The two formulas given on the drawing will apply to all cases generally by paying proper attention to signs, which are selected by inspection; minus for obtuse corner and plus for acute corner.

Probably most cases of this kind deal with building walls where the thicknesses T_1 and T_2 are small. For the purpose of detailing steel joists and reinforcing bars, it has been found that results may be obtained very rapidly and with satisfactory precision by the use of a slide rule alone, using the table of sines and tangents given on the rule.



CONTRACT UNIT PRICES

What contractors are bidding on different kinds of construction work

RELIEF SEWER, BOSTON

OWNER: Commonwealth of Massachusetts, Metropolitan District Commission; Joseph P. Dever, chief engineer, sewer division.

PROJECT: Constructing section 106-A North Metropolitan relief sewer in Medford near Mystic River. Consists of 4,710 ft. of 8 ft. 6-in. by 8 ft. 6-in. semi-elliptical sewer and a 3-pipe 54-in. cast iron syphon about 342 ft. long. Stipulated contract time is 12 months. Transportation facilities by rail and highway are available. The sewer line is practically all in open marsh land; it crosses one highway and one small river.

BIDS: Six bids were received Oct. 20, 1938, ranging from the contract low of $318,157 to $347,840. Minimum wage rates are as follows: skilled, $1.00 per hr.; semi-skilled, 75 c. per hr.; common, 65 c. per hr.

LOW BIDDERS:

1. Edward M. Matz, Inc., Jamaica Plain, Mass. (contract) $318,157
2. P. De Cristoboro, Roslindale, Mass..... 332,065
3. A. Baruffaldi, Somerville, Mass..... 336,409

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Earth excav. in trench above sewer grade	40,000 c. y.	$1.40	$1.00	$1.30
2. Earth excav. in trench below sewer grade	8,000 c. y.	.50	1.00	.50
3. Earth excav. for c. i. pipe siphon	6,000 c. y.	2.00	2.50	1.50
4. Earth excav. uncl.	1,000 c. y.	.50	1.00	.50
5. Gravel refill	8,000 c. y.	.50	1.50	.50
6. Rock excav.	200 c. y.	.50	2.00	1.30
7. Conc.	8,500 c. y.	10.50	12.00	15.00
8. Reinf.	300 tons	80.00	65.00	60.00
9. Pipe underdrain	6,000 l. f.	1.00	1.00	.50
10. Brick masonry	30 c. y.	40.00	30.00	35.00
11. Revetment pav.	2,000 s. y.	.50	.25	1.50
12. Lumber, sheeting used left in place	750 M.b.m.	35.00	30.00	30.00
13. Lumber, sheeting used, removed	550 M.b.m.	10.00	10.00	5.00
14. Struct. timber, for pile platform	200 M.b.m.	10.00	10.00	30.00
15. Steel sheeting used and left in place	500 s. f.	1.40	1.00	.70
16. Steel sheeting used and removed	24,500 s. f.	.60	1.50	.90
17. Minor drains	200 l. f.	2.00	1.00	1.00
18. Wood piles	50,000 l. f.	.25	.10	.25
19. Pile load tests	10 ea.	10.00	10.00	50.00
20. C. I. pipes and fittings	600 tons	75.00	70.00	65.00
21. Manhole frames and covers	3,600 tons	.06	.08	.03
22. Misc. iron	9,000 lb.	.06	.08	.03
23. Street pav. resurfaced	2,000 s. y.	.50	.25	.60
24. Grout	2,000 bags	.75	.40	.60
25. Prep. at site cleaning up, etc.	L. S.	5,000.00	5,000.00	500.00

HIGHWAYS, NEVADA

OWNER: Nevada Department of Highways; Robert A. Allen, state engineer.

PROJECT: Construction of 5.216 mi. of bituminous road-mix highway on Route 1 from Verdi to Lawton's. Minimum wage rates are as follows: skilled, $1.00 per hr.; semi-skilled, 70 c. per hr.; common, 62½ c. per hr.

BIDS: Five bids were opened Sept. 29, 1938, ranging from the contract low of $273,691 to $313,362.

LOW BIDDERS:

1. Isbell Construction Co., Reno (contract)... $273,691
2. Dodge Construction, Inc., Fallon, Nev..... 274,893
3. The Utah Construction Co., Ogden..... 288,408

Item	Quan.	Unit Prices (1)	(2)	
1. 75-hp. crawler type tractor	75 hr.	$6.50	$5.00	$5.00
2. 10-ft. blade grader	75 hr.	2.00	2.00	2.00
3. 75-hp. tractor with power scraper	75 hr.	7.50	7.00	7.00
4. Motor grader pneum. tires	150 hr.	5.00	5.00	5.00
5. 1½-ton truck	300 hr.	2.25	2.50	2.25
6. Spl. detours (set by engineer)	L. S.	4,825.00	4,825.00	4,825.00
7. Sel. detour surf.	8,686 ton	.50	.50	.50
8. Haul on sel. detour surf.	6,675 ton mi.	.10	.10	.50
9. Remove tread	47 ea.	5.00	10.00	20.00
10. Remove fence	35,331 l. f.	.05	.03	.02
11. Remove concr.	3 c. y.	10.00	10.00	10.00
12. Remove culv. pipe	677 l. f.	1.00	1.00	1.50
13. Remove culv. inlets and outlets	43 ea.	5.00	10.00	10.00
14. Remove bridge (lt. sta. 246+43)	L. S.	2,500.00	2,000.00	2,500.00
15. Remove bridge (rt. sta. 386+55)	L. S.	1,500.00	1,000.00	1,000.00
16. Remove temp. flume	L. S.	75.00	100.00	100.00
17. Remove concr. surf.	26,578 s. y.	.25	.35	.25
18. Roadway excav. "A"	25,014 c. y.	.96	1.00	1.35
19. Roadway excav. "B"	86,730 c. y.	.56	.55	.55
20. Drainage excav.	1,530 c. y.	1.00	.35	.60
21. "Y" type ditches	31 sta.	5.00	10.00	8.00
22. Borrow	12,676 c. y.	.35	.35	.35
23. Slope rounding	73 sta.	5.00	10.00	10.00
24. Overhaul	302,027 y. sta.	.01	.01	.01
25. Overhaul	24,525 y. mi.	.15	.12	.15
26. Struct. excav.	4,100 c. y.	2.25	2.00	3.25
27. Backfill	3,430 c. y.	.50	.50	1.25
28. Subgrade, type "B" (shoulders)	1.20 mi.	100.00	100.00	200.00
29. Subgrade type "B" (full width)	3.70 mi.	150.00	200.00	300.00
30. Rock cleanup	1,320 c. y.	1.50	1.00	1.50
31. Roadside cleanup (set by engineer)	L. S.	2,500.00	2,500.00	2,500.00
32. Water	3,931 M. gal.	1.50	1.50	1.50
33. Type 1 gravel base	30,488 ton	.65	.80	.65
34. Gravel surf.	18,614 ton	.80	.90	.80
35. Liquid asph. type MC-2 (seal)	27 ton	17.00	20.00	22.00
36. Blotter sand	51 ton	2.00	3.00	2.50
37. Liquid asph. type SC-2 (roadmix)	451 ton	17.00	16.00	17.00
38. Roadmix (13-ft. width)	1.20 mi.	250.00	250.00	300.00
39. Roadmix (22-ft. width)	3.70 mi.	500.00	600.00	600.00
40. Roadmix intersections	7 ea.	25.00	50.00	40.00
41. Cl. "A" conc.	891 c. y.	28.00	28.00	32.00
42. Cl. "B" conc.	255 c. y.	28.00	28.00	30.00
43. Cl. "D" conc.	314 c. y.	32.00	30.00	30.00
44. Conc. rail	387 l. f.	6.00	4.00	4.50
45. Reinf.	257,400 lb.	.05	.05	.06
46. Bronze bearing plates	1,100 lb.	.45	.50	.50
47. Untr. Douglas Fir	7.9 M.b.m.	80.00	100.00	125.00
48. 15-in. Corr. metal pipe (dipped)	168 l. f.	2.00	2.00	2.00
49. 18-in. Corr. metal pipe (dipped)	1,270 l. f.	2.50	2.25	2.25
50. 24-in. Corr. metal pipe (dipped)	514 l. f.	3.50	3.00	3.25
51. 30-in. Corr. metal pipe (dipped)	74 l. f.	4.00	3.50	4.50
52. Relay culv. pipe	378 l. f.	1.00	1.00	1.50
53. 8-in. Perf. underdrain	1,282 l. f.	1.00	1.25	1.75
54. Drain backfill	504 c. y.	2.50	2.00	2.00
55. Grouting drain backfill	1,380 l. f.	.40	.50	1.50
56. Weephole ext.	33 ea.	3.50	3.00	3.00
57. Grouted hand-laid riprap	9 c. y.	15.00	15.00	15.00
58. Move pipe, culvert, headwalls	35 ea.	10.00	10.00	10.00
59. Cl. "A" conc. curb and gutter	105 c. y.	25.00	20.00	20.00
60. Castings	2,150 lb.	.10	.15	.20
61. Beam type metal guard rail	2,450 l. f.	1.50	1.20	1.40
62. Culv. markers	92 ea.	3.50	3.50	4.00
63. Guide posts	15 ea.	3.00	3.00	4.00
64. Const. fence	1,234 l. f.	.20	.20	.15
65. Reconstruct fence	35,321 l. f.	.15	.10	.08
66. 16-ft. steel gates	9 ea.	20.00	40.00	25.00
67. Monuments	68 ea.	3.50	3.00	4.00
68. Metal flume	170 l. f.	2.00	2.00	4.00
69. Miscel. items	L. S.	100.00	250.00	150.00
70. Cyclopean conc.	777 c. y.	16.00	16.00	19.00
71. Stab. shoulders	7,172 s. y.	.10	.10	.08

DISTRIBUTION SYSTEM, UTAH

OWNER: City of Ogden, Utah; John C. Brown, city engineer.

PROJECT: Furnishing and installing 15,344 ft. of C.I. pipe and fittings varying in size from 20 to 6 in., for ex-

(*Contract Unit Prices continued on second following page*)